



STS Consultants Ltd.
Consulting Engineers

Waterfront Facilities Inspection

Naval Training Center
Great Lakes, Illinois

Department of the Navy

REPORT

MASTER LIST

" WATERFRONT FACILITIES INSPECTION "

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INSPECTION VIDEO TAPES



STS Consultants Ltd.
Consulting Engineers

111 Pfingsten Road
Northbrook, Illinois 60062
(312) 272-6520

July 29, 1988

Mr. Fred Estilo
Department of Navy
Navy Public Works Center, Building 1A
Great Lakes, IL 60088

RE: Contract N-62472-87-C-7706 - WATERFRONT FACILITIES INSPECTION

Dear Mr. Estilo:

STS Consultants, Ltd. has completed a waterfront facilities inspection for the Naval Training Center in Great Lakes, Illinois. Enclosed are twelve copies of the final report summarizing the inspection and study results, and a video tape of the underwater inspection areas of interest.

The project includes a condition assessment for waterfront facilities at the Naval Training Center. Above and below water observations of nearshore facilities were performed as part of this assessment. Furthermore, a structural analysis was performed to identify allowable structural loads on docks and piers along the waterfront. Finally, conceptual remedial measures were developed for facilities requiring attention.

It has been a pleasure to work with you on this project. Please feel free to call if we may be of further assistance.

Respectfully,

STS CONSULTANTS, LTD.

William J. Weaver, P.E.
Vice President, Water Resources Group

Linda M. Burke, P.E.
Project Engineer

WJW/th

1137-I

encl.

Report

Project

WATERFRONT FACILITIES INSPECTION
FOR THE
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

Client

DEPARTMENT OF THE NAVY
NAVY PUBLIC WORKS CENTER, BUILDING 1A
GREAT LAKES, ILLINOIS 60088

Project #

1137-I

Date

JULY 29, 1988



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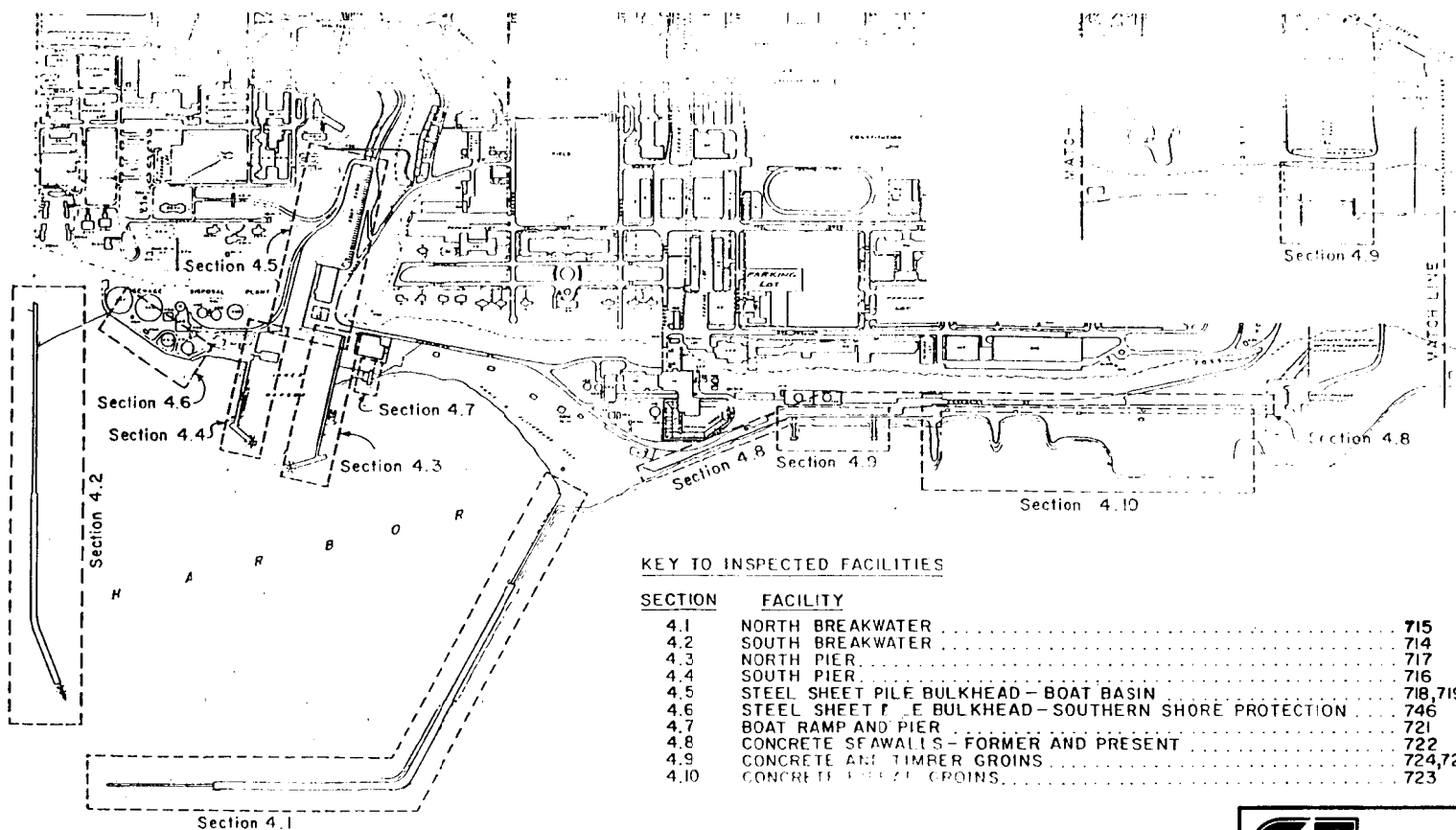
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EXECUTIVE SUMMARY

A condition assessment was performed for ten (10) waterfront facilities located along the 1.5 mile stretch of shoreline at the Naval Training Center (NTC). The location of these structures is illustrated on Figure 1, and a description of each facility is provided on Table 1. The NTC waterfront facilities were last inspected in 1980 (Reference 1). The project effort summarized herein includes an evaluation of how the facilities have deteriorated during the past seven years. The estimated useful life remaining for each of these facilities and the structural capacity of load bearing structures were evaluated. Areas determined to be in need of repair were identified and remedial measures were recommended. Opinions of probable cost are provided for each recommended repair item.

The project included a detailed above and below water visual observation of the condition of shoreline facilities. Non-destructive testing was performed in selected areas to enhance the condition assessment.

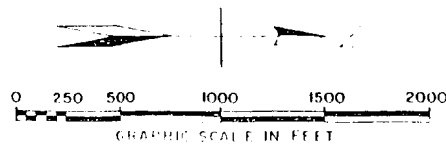
The report is complementary to the following two reports also being performed under contract N-62472-87-C-7706: 1) a Lake Michigan water level study (Reference 2), and 2) a comprehensive slope stability and erosion study (Reference 3). These two reports and the information provided herein provide a comprehensive evaluation of near-shore facilities. Table 2 summarizes the condition assessment including recommended remedial measures, and opinions of probable cost.



KEY TO INSPECTED FACILITIES

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LAKE MICHIGAN



NOTE: STATIONING USED ON ALL DRAWINGS IN THIS REPORT HAS BEEN ESTABLISHED BY CHILDS ENGINEERING CORPORATION.
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STS Consultants Ltd.
Consulting Engineers

KEY TO INSPECTED FACILITIES
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

DRAWN BY G.R.S.	DATE 5-88	SCALE SHOWN	STS PROJECT NO. 1137-1
CHECKED BY L.M.B.	DATE 6-88	SHEET NO. 1	STS FILE NO.

TABLE 1

NAVAL BASE
GREAT LAKES, ILLINOIS

WATERFRONT FACILITY SUMMARY TABLE

<u>Facility</u>	<u>Year Built</u>	<u>Length of Facility</u>	<u>Structural Type</u>
North Breakwater	1923	3,676' ±	1) Stone-filled concrete caissons - 540' 2) Pile-type rubble mound breakwater with limestone capping - 1512' 3) Pile-type rubble mound breakwater with granite capping - 578' 4) Stone-filled crib with concrete cap - 1046'
South Breakwater	1923	2,010' ±	1) Stone-filled concrete caissons - 108' 2) Pile-type rubble mound breakwater with limestone capping - 90' 3) Stone-filled crib with concrete cap - 852' 4) Concrete block retaining wall - 90'
North Pier	1919 (modified 1933)	800' ±	1) Stone-filled timber crib with stone filled concrete capping - 700' 2) Stone-filled timber crib with concrete retaining wall cap - 100'
Steel Sheet Pile Bulkhead - Boat Basin	Steel Sheet Piling Installed in 1945, 1953 & 1954	2,060' ±	Anchored steel sheet pile bulkhead with concrete deck
Steel Sheet Pile Bulkhead - Southern Shore Protection	Unknown	525' ±	Cantilever steel sheet pile bulkhead
Boat Ramp & Pier	1972 1984	60' ±	Concrete plank ramp and timber pier
Concrete Seawalls - Former & Present	Unknown	3,100' ±	Cast-in-place concrete, precast concrete, stone and concrete riprap
Concrete & Timber Groins	Unknown	480' ±	Precast concrete elements, timber piles
Rubble Groins	Unknown	700' ±	Fill and Concrete riprap

TABLE 2

**WATERFRONT FACILITY REMEDIAL REPAIRS
RECOMMENDATION SUMMARY**

<u>Opinion of Probable Cost</u>				
<u>Facility</u>	<u>Recommendations</u>	<u>Materials</u>	<u>Labor</u>	<u>Opinion of Probable Cost</u>
4.1) North Breakwater: 715				
a) Concrete Caisson	- Repair deteriorated concrete cap	\$ 11,500	\$ 26,500	\$ 38,000
	- Repair concrete surface deterioration	3,250	15,750	19,000
b) Limestone Rubble Mound	- Monitor stability			----
c) Granite Rubble Mound	- No repairs at this time			----
d) Timber Pile Crib	- Encase in stone revetment	36,500	65,500	102,000
	- Repair concrete surface deterioration	5,800	28,200	34,000
4.2) South Breakwater: 714				
a) Concrete Caisson	- Repair concrete surface deterioration	1,100	4,900	6,000
b) Limestone Rubble Mound	- Monitor voids and stability periodically			----
c) Timber Pile Crib	- Encase failed portion in revetment	22,500	42,500	65,000
	- Repair concrete surface deterioration	3,300	15,700	19,000
4.3) North Pier: 717				
a) Outer Leg	- Repair pier end/concrete cap	45,300	41,700	87,000
b) Main Pier	- Repair concrete surface deterioration	6,900	33,100	40,000
	- Add stone to overdredged areas	11,600	4,400	16,000
c) Retaining Wall	- No repairs at this time			----
4.4) South Pier: 716				
a) Outer Leg & Main Pier	- Repair concrete surface deterioration	4,500	21,500	26,000
b) Retaining Wall	- Repair timber curbing	1,300	2,200	4,500
4.5) Boat Basin Seawall: 720				
	- Recap upper concrete wall, replace pipe railing, replace timber railing	32,100	77,900	110,000
4.6) South Seawall: 746				
	- Fill voids and cap deck with pavement	4,000	1,300	5,300
4.7) Boat Ramp: 721				
	- No repairs at this time			----

TABLE 2 (con't)

**WATERFRONT FACILITY REMEDIAL REPAIRS
RECOMMENDATION SUMMARY**

		<u>Opinion of Probable Cost</u>		
<u>Facility</u>	<u>Recommendations</u>	<u>Materials</u>	<u>Labor</u>	<u>Opinion of Probable Cost</u>
4.8) North Seawall: 722	- Place 100 ft of rubble mound revetment	\$ 12,000	\$ 23,000	\$ 35,000
	- Repair undermined section of wall	300	700	1,000
	- Repair deteriorated concrete wall top	3,000	8,000	11,000
4.9) Concrete Timber Groins: 724, 725, 726, 727				
a) Concrete Groins	- Structures have failed/no repairs warranted			----
b) Timber Groins	- Repair toe protection by placing a stone revetment	151,400	88,600	240,000
	- Flatten slope/reshape to 2.5:1.0	35,300	20,700	56,000
	- Perform slope stability assessment			----
4.10) Riprap Groins 723	- No immediate critical repairs required			----
	- If repair of minor erosion is desired, place a stone revetment	\$ 35/l.f.	\$ 65/l.f.	\$ 100/l.f.
	- Monitor condition periodically			----

**WATERFRONT FACILITIES INSPECTION
GREAT LAKES NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS**

1.0 INTRODUCTION

The Naval Training Center (NTC) at Great Lakes, Illinois encompasses a shoreline area of approximately 1.5 miles along Lake Michigan. Over the years, the shoreline has been modified extensively with the construction of breakwaters, groins, jetties, revetments and seawalls. The Naval Public Works Center commissioned STS Consultants, Ltd. (STS) to perform a comprehensive waterfront facilities inspection to assess the condition of existing nearshore structures, and to identify conceptual remedial measures for problems which are identified.

The study included an investigation above and below the water line at ten (10) designated waterfront facilities. The location of each is illustrated on Figure 1, and a list and description of these facilities is provided on Table 1 in the Executive Summary. These facilities were last inspected in 1980 (Reference 1). A description of the subtasks comprising the STS study effort follows:

- A. Data Collection and Mobilization - Available site plans, data, maps, construction plans, aerial photographs, topography, soil boring data, and previous reports and studies were obtained from the NTC at the initial project mobilization meeting. Record drawings and site plans (Reference 4) provided by the NTC served as base maps for this study. These maps were prepared as part of the 1980 waterfront facilities inspection and were updated as part of this project to illustrate changed conditions.
- B. Land Survey - A survey was performed to supplement and update existing information. Stationing was reestablished for all designated shore structures to be compatible with the stationing system which was utilized in the past. A cross section survey

of designated structures at 21 locations was performed. Sounding surveys at the toe of all offshore structures was performed. Finally, an alignment and elevation survey was performed for the concrete seawall which exists north of the NTC harbor.

- C. Above Water Site Investigation - Visual condition investigations were performed in conjunction with field surveys for all designated waterfront protection structures to identify the location and extent of structural defects. Photographs were taken at selected accessible locations to document the physical conditions of each structure. The primary objective of these investigations was to identify settlement cracks, holes, separations, eroded surfaces, material failures, and any other significant and unusual physical defects.
- D. Underwater Investigations - Underwater observations were performed to assess facility conditions below the water line for selected offshore structures. The physical condition of these structures was evaluated based upon this observation.
- E. Non-Destructive Testing - Non-destructive testing was performed at selected locations to evaluate the condition of various shore structure materials. Subsurface interface radar surveys were performed on the south boat pier and the south seawall to investigate the potential existence of voids under deck surfaces. An ultrasonic survey was performed to estimate steel thickness measurements for the south seawall.
- F. Structural Analysis - A structural analysis of selected waterfront structures was performed based upon the record data and information obtained during the site investigation. Estimated permissible loads that could be superimposed on piers and dockwalls with roadways adjacent were evaluated. Allowable uniform and vehicular live loads were also estimated. Finally, the life expectancy of each waterfront

structure was estimated based upon existing conditions. Please note that the structural analysis results represent best possible estimates given the available site data. Changed conditions or unusual substructures or subsurface conditions could modify the analysis results.

- G. Development of Conceptual Remedial Measures - Conceptual remedial measures for structures in need of repair were developed.
- H. Opinion of Probable Cost - An opinion of probable cost was developed for each of the conceptual remedial measures that was developed. A detailed breakdown of quantities and material costs is provided for each alternative. Furthermore, the costs have been classified among repairs, reconstruction and new construction items.

This report is complementary to the following two reports also being performed under Contract N-62472-87-C-7706: 1) a Lake Michigan water level study (Reference 2), and 2) a comprehensive slope stability and erosion study (Reference 3). These two reports identify dynamic water conditions and the condition of structures adjacent to Lake Michigan and the Pettibone Creek. Together, these two reports and the information provided herein provide a comprehensive evaluation of erosion, and structure and slope stability conditions.

2.0 DATA COLLECTION

Data and information available from the NTC and other sources were obtained at the project outset. Furthermore, surveys were performed to provide data necessary to draw conclusions for the waterfront facility evaluation. Following is a summary of the data collection effort:

- a) The U.S. Navy Public Works Center drawing files were reviewed to develop the history of construction and rehabilitation for shore protection structures at the NTC.
- b) The final report for the most recent waterfront facilities inspection performed at the NTC in 1980 was obtained and reviewed (Reference 1).
- c) STS project files were researched to review soil information in the harbor and waterfront area.
- d) Topographic maps covering the NTC waterfront were obtained from the Naval Public Works Office. The maps are drawn to a scale of 1" = 50' with one foot contour intervals (Reference 4).
- e) Mr. Ed Chambliss, the NTC Harbormaster, was interviewed to obtain information pertaining to the recent history and performance of the harbor structures.
- f) Typical cross sections of each structure were surveyed to obtain elevation data. The survey results were referenced to the Great Lakes Naval Training Center Datum (NTCD), which is 580.912 feet above mean tide at New York Harbor. Elevations presented in this report are referenced to NTCD unless otherwise noted. The

conversion from NTCD to two other common elevation references, International Great Lakes Datum (IGLD) and National Geodetic Vertical Datum (NGVD) is presented on Table 3.

Horizontal control was established by measuring and stationing each waterfront structure. Marks placed every 50 feet enabled the observer to note his relative location when making observations. Where possible, the stationing was established in accordance with the system used in the past (Reference 1).

- g) Depth soundings were obtained by Hydrographic Survey Company under subcontract to STS. Toe of structure elevations were obtained at a uniform 35 ft from the structure centerline. Soundings were measured relative to the Lake Michigan water level. Hourly lake level readings were obtained at the Calumet Harbor and Milwaukee NOAA lake level gages. Sounding depths were then adjusted to reference IGLD and the NTCD.
- h) The alignment of the concrete seawall was surveyed by measuring, at 20 foot intervals, the offset from a baseline established approximately parallel to the structure.

Specific results of the land survey are presented in the detailed discussion for each waterfront structure.

TABLE 3
DATUM CONVERSION FACTORS

<u>Elevation Given in Datum</u>	<u>Add to Convert to Datum</u>		
	<u>MSL</u>	<u>IGLD</u>	<u>NTC</u>
MSL	----	-1.4	-580.6
IGLD	+1.4	----	-579.2
NTC	+580.6	+579.2	----

3.0 METHODOLOGY

Examination of waterfront facilities at the NTC was accomplished through above and below water observations of structural conditions, and selected non-destructive testing of materials. Above water tasks were performed by members of STS Consultants, Ltd.'s engineering staff. Observations and tests were performed by or under the supervision of a registered professional engineer. Underwater observation and data collection tasks were performed by Hydrographic Survey Company (HSC). These tasks included bathymetric measurements, side scan sonar surveys, and visual observation.

This study provides a general structural assessment and identifies needed repairs. The level of investigation performed was limited to visual and tactile observation. Non-destructive testing methods were utilized for selected structures to enhance the evaluation. The results of this study provide a general evaluation of the condition of selected waterfront facilities based upon the methods employed. Other problem areas may exist which cannot be readily identified by the methods employed. Therefore, the facilities should be observed periodically to evaluate changed conditions. Following is a summary description of each evaluation technique employed:

- a) Visual Observation - Visual observation was the primary method used to identify problem areas. Above water observations were recorded on a hand-held tape recorder and later transcribed. Numerous photographs were taken of each waterfront structure. Below water visual observations of the breakwaters and piers were performed by the HSC dive team using an underwater communications system. A portion of the underwater observations was recorded on video tape to document the condition of several areas of interest. A copy of this tape has been furnished to the Naval Public Works Office.

- b) **Impact Testing** - A relative measure of concrete quality can be obtained by striking a concrete surface with a hammer and comparing the rebound and sound of the hammer strike. Concrete which is severely delaminated will make a dull "thud" sound when struck, and the hammer will not rebound. Concrete in good condition will make a crisp, sharp sound when struck and will cause the hammer to rebound equally as hard as it was struck. Comparative results can be obtained by rating the response to impact on a scale of one to three (best to worst). Impact testing is a quick and simple method to obtain rule of thumb results when destructive testing is not feasible.
- c) **Measurements - Dimensions** were obtained by one of three methods: direct measurement by tape or ruler, estimated measurement by pacing, and measurement by reference to station. Stationing was established to be the same as the previous waterfront inspection performed in 1980 (Reference 1).
- d) **Void Detection** - Subsurface Interface Radar (SIR) exploration and concrete cores were used to investigate the potential presence of voids on the north pier and south seawall.
- e) **Steel Thickness** - Ultrasonic testing was used to estimate steel sheet pile thickness on the south seawall.
- f) **Test Excavation** - A test excavation was performed adjacent to the south seawall to investigate the potential existence of a tieback system. The test excavation extended 5.5 feet deep, and was performed using a tractor-mounted backhoe.

A detailed description of non-destructive testing methods utilized for this project follows. Testing results are provided herein.

3.1 Subsurface Interface Radar (SIR) Exploration

A subsurface interface radar (SIR) survey was conducted on the main pier and along the sheet piling deck north of the harbor to explore for indications of voids beneath the concrete slabs. The SIR system utilized was a GSSI SIR-3 with a profiling graphic recorder and a Model 3102 transducer operating at 500 MHz, 2.0 nanosecond pulsewidth, with a range setting of 40 to 60 nanoseconds. The SIR operates by transmitting a high frequency electromagnetic impulse into the subsurface. This signal penetrates the subsurface and is reflected off buried dielectric interfaces back up to the receiver. These reflected signals are printed on the graphic printer.

Different materials affect the dielectrical reflection and transmission properties of the SIR signal. Concrete, steel, sand, clay, void spaces and fill materials have dissimilar dielectrical properties and hence the interface between these materials provides a strong contrast on the SIR record. Generally, clays and saturated materials, in contrast to sands and unsaturated materials, tend to absorb or attenuate SIR signals thereby reducing the effective signal penetration. The presence of a void may be manifested on the SIR record as a highly reflective zone representing the boundary between the void below the bottom of the slab and the upper boundary of the fill material. It is doubtful the SIR signals penetrated the strongly reflective water surface within the piers and sheet piling wall.

The transducer was moved over the surface of selected concrete pier and deck areas. Numerous passes using different range and signal gain settings were made until a satisfactory SIR record was obtained. The position of each scan was noted in the field on the SIR record. Three scan lines were performed along the major axis and several perpendicular scans were taken at regular intervals across each pier or deck survey area. Anomalous SIR results were noted in the field during data acquisition and several anomalous zones were surveyed in more detail with additional SIR scans.

Although SIR is commonly used to located buried objects, such as voids, piping, excavations, and other structures, the results are interpreted and judgment is required. The positive correlation between the interpreted results and the actual conditions gives confidence in the interpretations. This report represents our engineering judgment and no warranty, either expressed or implied, is contained herein.

A SIR survey was performed in November, 1987 across the surfaces of the north pier and the sheet pile seawall to explore the potential presence of voids beneath the concrete slabs covering these areas. Based on an interpretation of the SIR records, there were a number of zones where voids were possible. Coring through the slabs within these zones was necessary to confirm the SIR record interpretations. Cores were also conducted in areas where no voids were expected.

Concrete coring was performed on January 18, 1988 at eight locations. The coring was conducted at locations selected on the basis of results obtained from the Subsurface Interface Radar Survey (SIR). Three cores were performed on the outer north pier, three on the main north pier and two were obtained from the concrete slab located landward of the south steel seawall.

3.2 Steel Thickness Measurements

Two portions of the NTC harbor facilities include steel sheet pile walls at the waters edge: The south steel sheet pile seawall (Section 4.6), and the boat basin (Section 4.5). Steel thickness measurements were obtained for the steel wall in the boat basin during the 1980 waterfront facilities inspection (Reference 1). Results reported from that effort indicated negligible deterioration of the steel wall. No noticeable deterioration of the steel at the seawall was noted by STS during the site observations in the Fall of 1987.

Since the boat basin walls were measured within a seven year period, new steel thickness measurements were not performed for this report. However, thickness measurements were performed for the steel seawall in Section 4.6 which was not measured in 1980.

The south seawall steel sheet piling thickness was measured using a Krautkramer Model DM-2 ultrasonic thickness gage. Ultrasonic gages can measure the thickness of many materials simply, accurately, and rapidly from only one side of the part. They use a pulse-echo technique which is similar to sonar, sending a sound pulse which travels through the material, bounces off the back surface, and travels back through the part. The gages measure the transit time, compensate for the correct sound velocity of the material, and present a digital readout of thickness. The operator merely places the transducer in contact with the part, using a liquid couplant, and instantly reads the thickness on the digital display.

4.0 WATERFRONT FACILITY CONDITION ASSESSMENT

The condition of selected waterfront facilities along the Lake Michigan shoreline at the NTC was performed based upon a combination of visual observations, testing, and engineering computations. Visual investigations were performed in October and November of 1987. The condition assessment includes an identification of the location and extent of structural defects for ten selected facilities. The above water and below water observations were performed to identify settlement cracks, holes, separations, eroded surfaces, material failures, and other significant or unusual physical defects.

Material testing which was performed to assist in the evaluation included the following: backhoe excavations to evaluate the existence and condition of tierods, side scan sonar to indicate structural deterioration underwater, bathymetric surveys, ultrasonic testing for selected seawalls, subsurface interface radar surveys, and concrete coring. Underwater video recordings and above water photographs were prepared to document this condition assessment.

Presentation of the condition assessment is subdivided into ten subsections in this Chapter. Each subsection addresses a different waterfront facility:

<u>Section</u>	<u>Facility</u>
4.1	Breakwater
4.2	South Breakwater
4.3	North Pier
4.4	South Pier
4.5	Steel sheet pile bulkhead - boat basin
4.6	Steel sheet pile bulkhead - southern shore protection
4.7	Boat ramp and pier
4.8	Concrete seawalls - former and present
4.9	Concrete and timber groins
4.10	Concrete riprap groins

The condition assessment for each waterfront facility includes a discussion for each of the following:

- Above water inspection results
- Below water inspection results
- Non-destructive testing (if applicable)
- General condition assessment and life expectancy
- Structural capacity (if applicable)
- Remedial measures

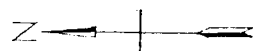
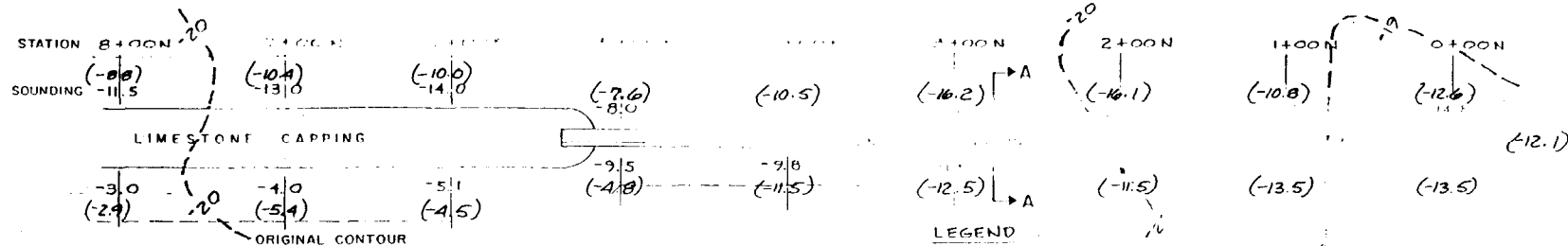
A subchapter is devoted to each of the ten designated waterfront structures. These subchapters include a general description of the facility, graphic representations including a plan view and cross-sectional details, photographs of selected problem areas, observations, and recommendations.

4.1 North Breakwater

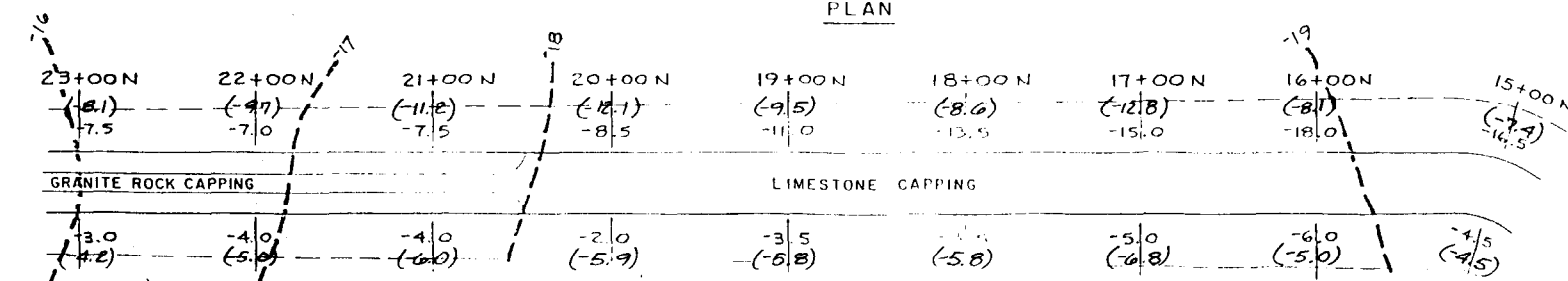
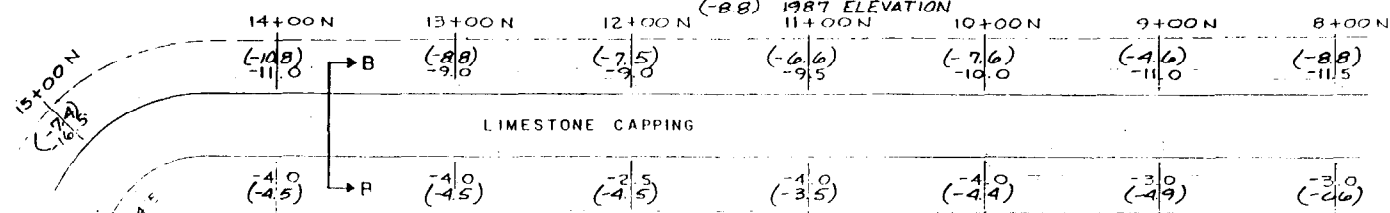
4.1.1. Description

The North Breakwater is located near the NTC Power Plant (Building 12). Figure Nos. 2A and 2B illustrate the plan view configuration and stationing system for this structure. The breakwater bears approximately S 62° E for 1724 feet, then proceeds due south for 1491 feet. The structure was built in 1923 and is comprised of the following four cross section configurations:

- a) Concrete Caisson - The outermost 540 feet of the breakwater is constructed of stone filled reinforced concrete caissons with reinforced concrete capping. This portion of the breakwater is comprised of 10 concrete caisson sections, each 54 feet long. The concrete capping was placed in 2 sections per caisson. A typical cross section is illustrated in Figure 3. The 2nd caisson from the harbor mouth supports a beacon tower which flashes a red harbor light.
- b) Limestone Rubble Mound - The next 1512 feet of the breakwater (station 5+40 to 20+52) is a Bedford limestone rubble mound. This section was constructed by cutting limestone pieces approximately five ft by five ft by ten ft. These pieces were placed with the long edge parallel to the breakwater axis. The stone placement is illustrated on the cross section presented in Figure 4.
- c) Granite Rubble Mound - The length of this section is 578 feet (station 20+52 to 26+30). The granite is angular stone which was placed randomly. A typical cross section is shown on Figure 5.
- d) Timber Pile Crib - The near-shore end of the breakwater is a rock-filled timber pile crib capped with reinforced concrete. The timber crib was formed by driving two rows of closely spaced piles and connecting them by tie rods every eight feet.



PLAN



NOTE:

SOUNDINGS AND CONTOURS ARE GIVEN IN FEET BELOW NAVAL BASE DATUM OF 0.00'.

SOUNDINGS ALONG BREAKWATER WERE MEASURED 35' FROM CENTERLINE OF BREAKWATER

PLAN

GRAPHIC SCALE IN FEET

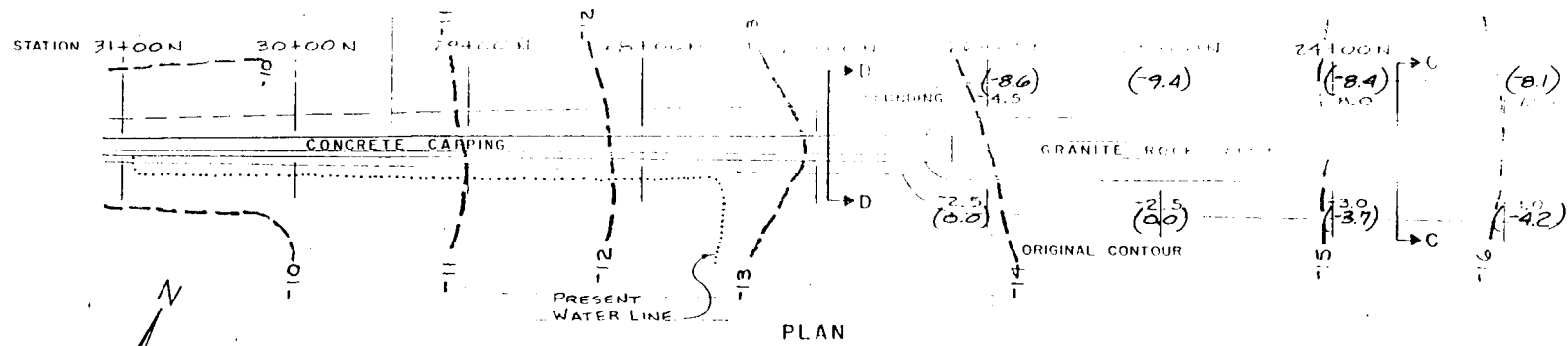


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FACILITY NO. 715
NORTH BREAKWATER PLAN
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

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L.M.B.	6-88	2A	



LEGEND

--- ORIGINAL CONTOUR (1923)

=7.5 1980 ELEVATIONS

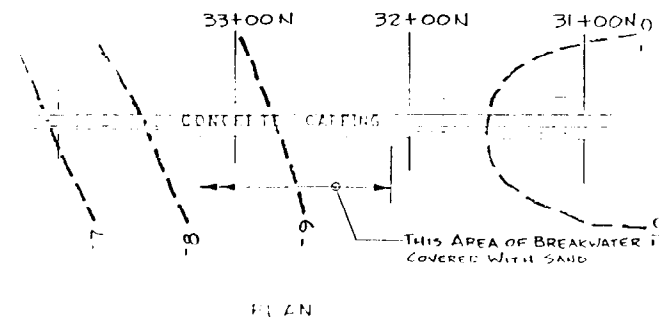
--- ORIGINAL TOE OF BREAKWATER (1923)

(-8.6) 1987 ELEVATION

NOTE:

SOUNDINGS AND CONTOURS ARE GIVEN IN FEET BELOW NAVAL BASE DATUM OF 0.00'.

SOUNDINGS ALONG BREAKWATER WERE MEASURED 35' FROM CENTERLINE OF BREAKWATER.

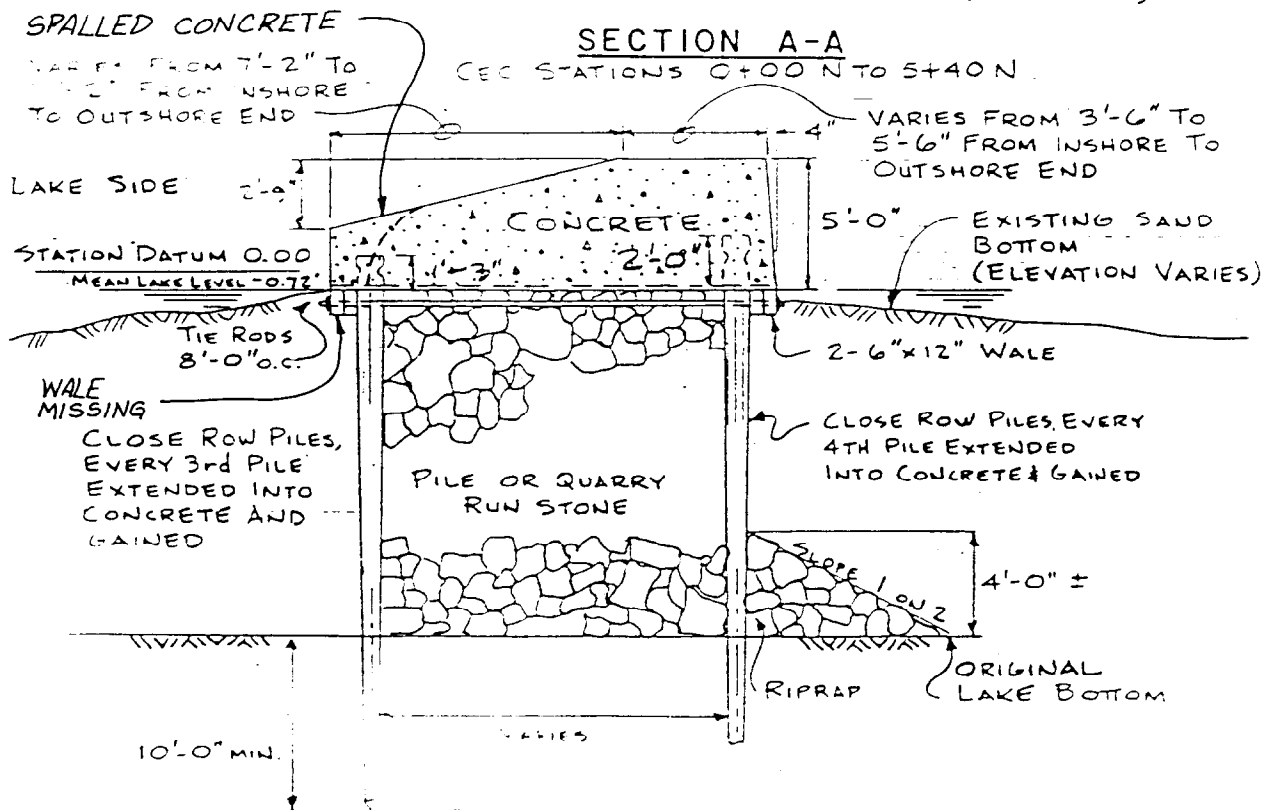
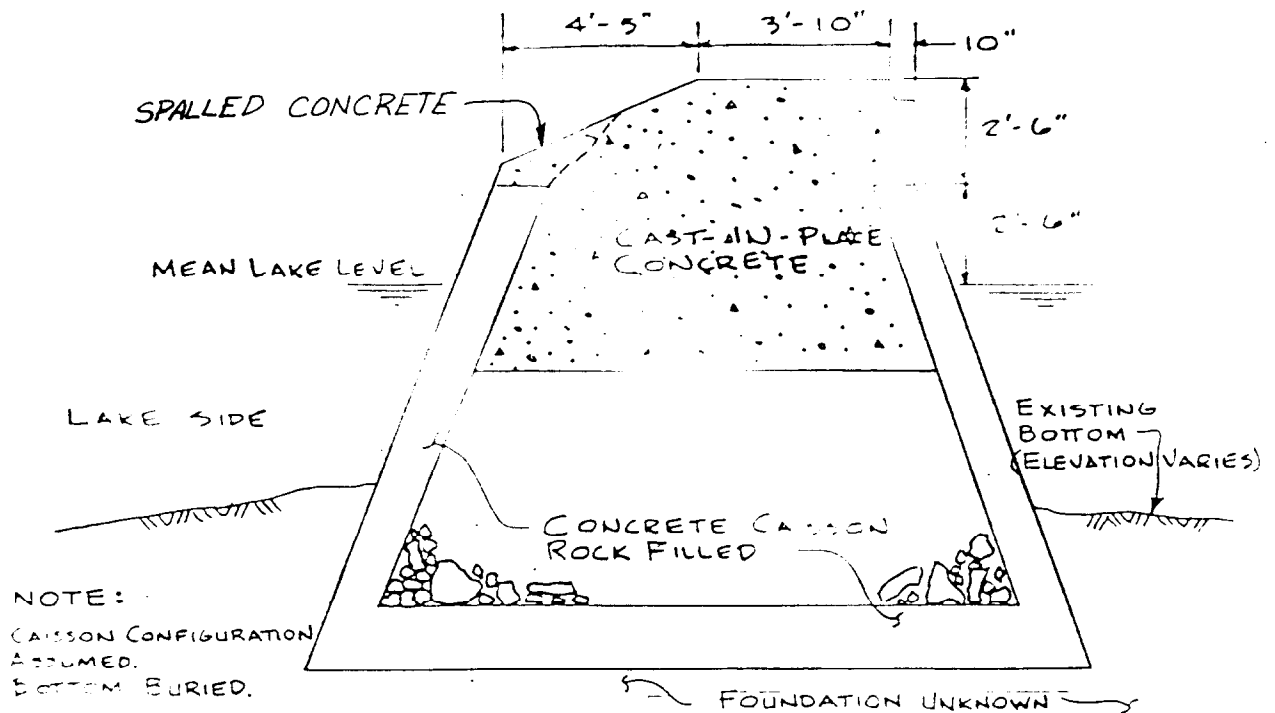


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NORTH BREAKWATER PLAN (CONT'D)
WATERFRONT FACILITIES INSPECTION
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L.M.B.	6-88	2B	



SECTION D-D

CEC STATIONS 26+20N TO INSHORE END

NOT TO SCALE

(CROSS-SECTIONS OF INSHORE & OUTSHORE CONCRETE SECTIONS)



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NORTH BREAKWATER SECTIONS
WATERFRONT FACILITIES INSPECTION
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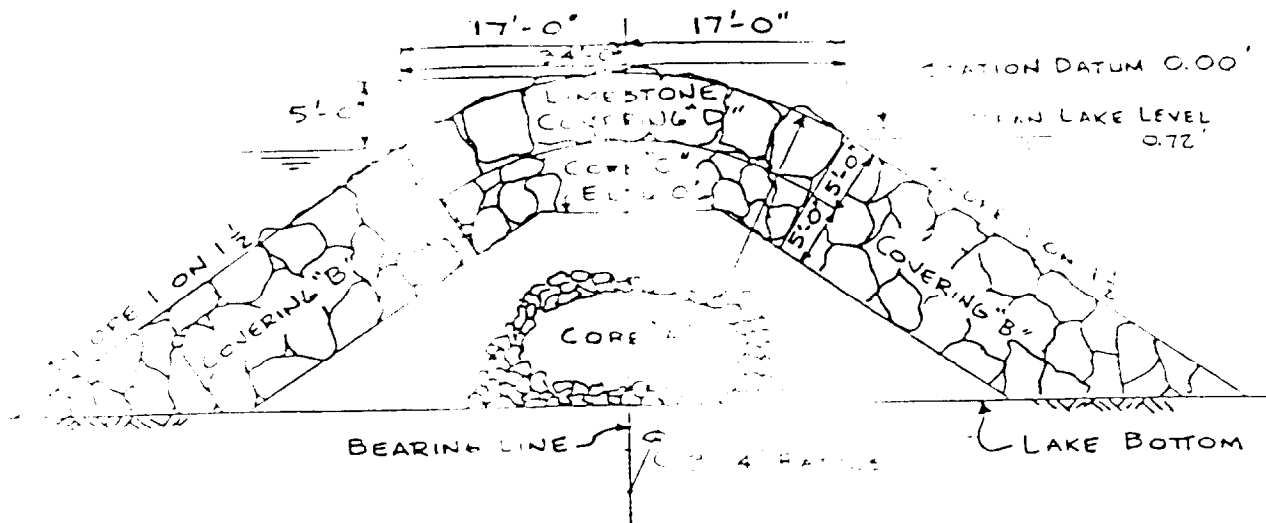
FIGURE NO

SHOWN

3

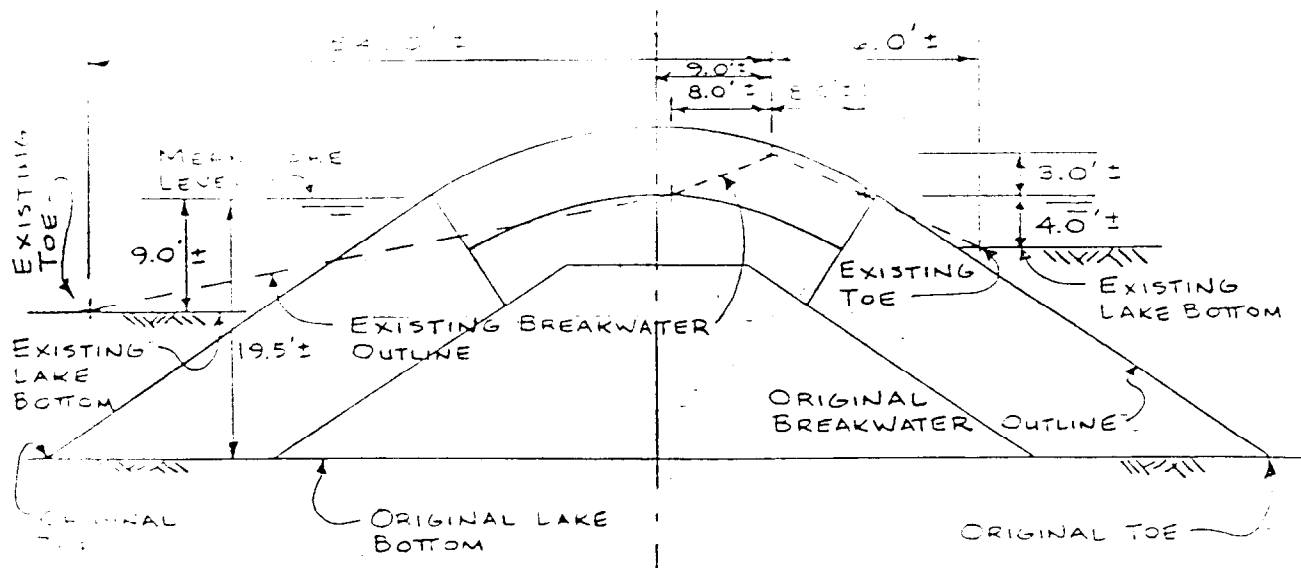
STS DRAWING NO

1137-1



SECTION E-B

ELEVATION OF ORIGINAL DESIGN
CEC STATIONS 5+40N TO 20+50N



CROSS-SECTION THROUGH CEC STATION 13+00N
SHOWING EXISTING BREAKWATER OUTLINE
SUPERIMPOSED ON ORIGINAL OUTLINE

(CROSS-SECTIONS OF LIMESTONE-
CAPPED PORTION OF THE RUBBLE
MOUND SECTION)

NOT TO SCALE



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FACILITY NO. 715
NORTH BREAKWATER SECTIONS
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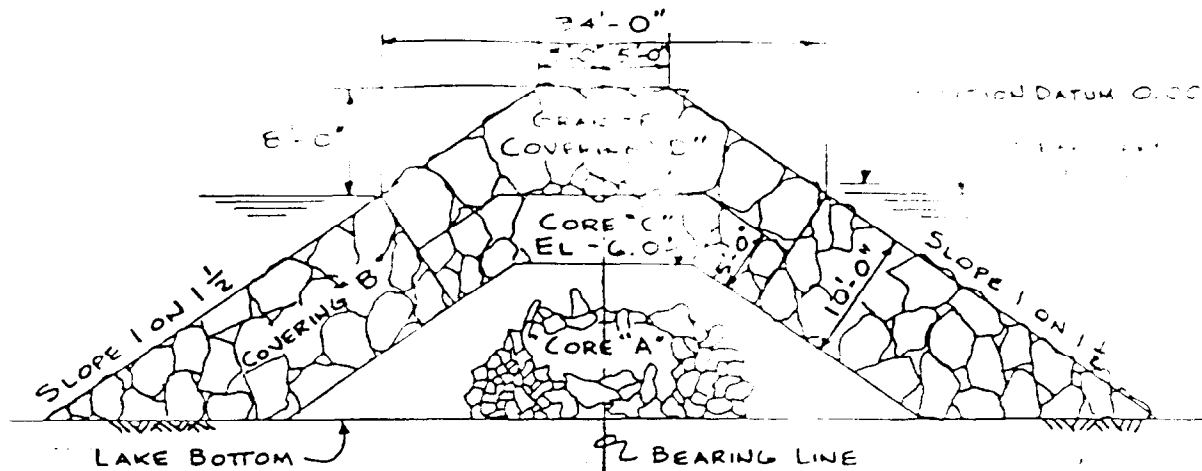
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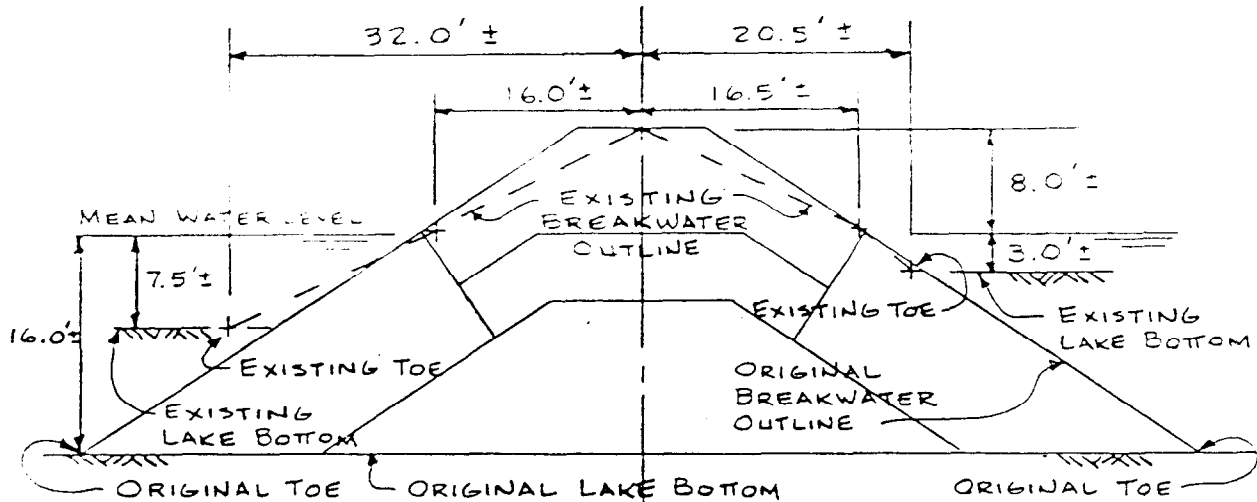
SCALE SHOWN FIGURE NO 4

STS DRAWING NO 1137-I



SECTION C-C

ELEVATION OF ORIGINAL DESIGN
CEC STATIONS 20+50 N TO 26+20 N



CROSS-SECTION THROUGH CEC STATION 23+00 N
SHOWING EXISTING BREAKWATER OUTLINE
SUPERIMPOSED ON ORIGINAL OUTLINE

(CROSS-SECTIONS OF GRANITE-
CAPPED PORTION OF THE
RUBBLE MOUND SECTION)

NOT TO SCALE



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PROJECT/CLIENT

FACILITY NO. 715
NORTH BREAKWATER SECTIONS
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

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SCALE
SHOWN

FIGURE NO

5

STS DRAWING NO.

1137-1

Twelve inch square timber wales were attached to each end of the tie rod on the outside of the row piles. Every 3rd close pile on the north side was extended one foot-three inches, and every 4th close pile on the south side was extended two feet. The concrete cap was cast in place around the extended piles and on top of the wale (Figure 3).

The concrete capped timber pile crib section was constructed to a length of 1046 feet extending to the Building 12A location. Only 584 feet of this section are visible at this time (Station 26+30 to 32+14). The remainder of the breakwater may have been removed since the original construction, or may have been buried by landfill or littoral drift deposition.

Inspection results are presented separately for each of the four breakwater configurations.

4.1.2 Visual Observations

- a) Concrete Caisson - The concrete caissons at the harbor mouth show some signs of deterioration. Spalling of concrete to depths of three to eight inches has taken place at most construction joints. Reinforcing steel is visible in many of these locations (Photo 5.07). The most severe concrete loss occurs on the caisson located between station 3+78 and 4+05. The caisson cap has completely eroded away on the east side of the structure. The top of the caisson base and the reinforcing steel are completely exposed (Photo 5.18). In general, approximately 20% to 30% of the concrete surface above the water line has experienced some deterioration.

The alignment of the caissons is good-less than 2° out of level. The structure changes less than 0.3 feet in elevation from the north caisson to the light beacon, with the north end being higher. The northernmost three feet of the north caisson



Photo 5.07 - Exposed reinforcing steel on east side of North Breakwater caisson.



Photo 5.18 - Top of North Breakwater caisson exposed due to spalling of concrete cap.

000006A014



Photo 2.24 - Settlement of limestone section of North Breakwater.



Photo 3.01 - Weathering of North Breakwater limestone has reduced average stone size.

concrete has severely delaminated. The remaining concrete is in good condition, especially considering that air-entrainment of concrete was not developed when the structure was built in the 1920's. Water depths adjacent to this structure appear to be similar to those measured in 1980. These depths range from 8 to 16 feet.

The underwater observations of the concrete caissons indicate that the severe concrete damage is limited to that above the present water level. Riprap was encountered at the base of the caissons from station 4+25 on the east side, around the harbor mouth, to 1+75 on the west side of the caissons. Randomly spaced hair-line cracks also exist as noted in the 1980 inspection report.

The beacon tower cubicle appears to be in similar condition to that which was noted in the 1980 inspection report. No major problems are apparent.

- b) Limestone Rubble Mound - The cut limestone section of the breakwater has settled considerably since the structure was built (Photo 2.24). A land survey of the limestone and caisson sections was not possible due to safety hazards, but estimates indicate that settlement during the past seven years has probably been minimal. Although the existing elevation is not significantly different from the original, the original cross section width has decreased dramatically. As discussed in the 1980 inspection report (Reference 1), the cross section has been altered over time due to differential settlement of the stones on the inside and outside of the harbor. Individually, few of the stones are intact. Most have failed along a plane parallel to one of the cut faces of the stone. In many cases, the stones are 1/2 to 1/3 of their original size (Photo 3.01).

Underwater, the limestone section of the breakwater was observed under limited (1-2 feet) visibility conditions. No voids in the armor stone were detected, and no leakage of core stone was found at the base of the structure with the exception of

the inside of the breakwater near section 15+00. At this location, smaller stone was encountered on the harbor bottom, but no source for the stone could be located along the structure face. The side scan sonar survey indicates a fairly uniform toe alignment for this structure.

As indicated in the 1980 inspection report, approximately 40% of the armor stones are disoriented on the lakeside of the breakwater. Large gaps exist between the stones which generally lean towards the harbor. At several locations, gaps up to several feet wide exist between the harbor stones which is exposing some of the underlayer materials. The width of the breakwater varies between 15 and 30 ft at the waterline. The smaller width sections generally occur where the majority of armor layer movement has occurred. The height of the breakwater above the lake level is also at a minimum at these sections. Water depths on the lake side of the structure appear to have reduced from two to eight feet over the past seven years. The harbor side of the structure has not changed significantly in terms of water depths.

The structure does not appear to have moved significantly since the 1980 inspection. The structural stability is difficult to evaluate due to the high variability and continued movement of the structure. Therefore, it is recommended that continued monitoring of the structure be performed on an annual basis.

- b) Granite Rubble Mound - The granite section does not appear to have experienced stone movement or settlement. No cracking or breaking of stones were detected. This section is in good condition. Water depths have generally remained unchanged over the past seven years. The cross section of this structure also appears unchanged. Underwater inspection results were the same as for the limestone rubble mound portions of this breakwater.

- c) Timber Pile Crib - The nearshore concrete capping exhibits two problem conditions:
- a) spalling and erosion of the concrete capping, leading to b) exposure and decay of the supporting timber piles. Severe spalling and delamination has occurred on the lakeward side of the concrete capping. In general, approximately 20% of the concrete surface area has spalled. This is probably due to ice damage during the recent high water levels. 'In some cases, almost one foot of concrete is missing from the north edge of the capping (Photo 3.12). The exposed timber piles and spalled concrete will experience accelerated decay due to the wet-dry cycles of wave action and storm surge. Surface spalling has occurred on the south edge of the capping, but this damage is more cosmetic in nature. A large void about two feet wide, two feet deep, and traversing the cross section of capping is apparent at the construction joint near station 27+50 (Photo 3.21). Water depths ranged from zero to nine ft along this structure.

The stone under the concrete capping appears to have been somewhat washed out from the timber crib leaving a one to two foot void below the concrete cap. The wales through which the tie rods were connected appear to have decayed or fallen off, and a majority of the tie rods are damaged by rust. The submerged portion of the timber piles are in satisfactory condition.

4.1.3 Recommendations

Following are recommendations for each of the four sections comprising the north breakwater:

- a) Concrete Caisson - The portion of the north breakwater consisting of concrete caisson is in fair condition considering its age. The spalling of the concrete continues unchecked and is slightly worse than that which was reported in the 1980 inspection report. The reinforcing in the concrete cap has been exposed and a



Photo 3.12 - Exposed timber piles on north side of North Breakwater capping section.



Photo 3.21 - Damaged capping section near North Breakwater station 27+50.

000006 A02Y

substantial amount of concrete has deteriorated. This deterioration can be expected to continue. Structural integrity of the concrete caisson portion is in question in several areas. These are the same areas that were outlined in the 1980 report as follows: a hole in the concrete cap and exposed steel reinforcement near Station 3+96 should be repaired. The opinion of probable costs to repair these two portions of the cap is \$38,000 (\$11,500 material, \$26,500 labor). The areas of the cap where spalling has exceeded a depth of 3 inches should also be repaired. Continued deterioration of these areas will result in structural integrity problems down the road. Repair of the concrete which is required on approximately 15% of the surface of the cap would be approximately \$19,000 (\$3,250 material, \$15,750 labor).

If the repairs are implemented as suggested, the life of the structure would be expected to last another ten years. A routine maintenance program would enhance the potential useful life even more. Without implementation of the suggested repairs, the life of the structure is difficult to evaluate and would depend upon the fluctuation of Lake Michigan water levels and the extent of wave activity.

- b) Limestone Rubble Mound - Deformation of this section over the past seven years appears to have been minimal. However, the original deformation which has occurred since the 1920s has been extensive. This structure should be watched closely. The movement of the structure has been highly variable and there is no exact way of knowing the extent of movement from year-to-year. Therefore, it is difficult to assess the structural condition of this section of the structure. Continued monitoring of this portion of the breakwater is recommended as part of normal maintenance procedures. No immediate repairs are recommended at this time.
- c) Granite Rubble Mound - This section is in good condition and is not in need of repairs at this time.

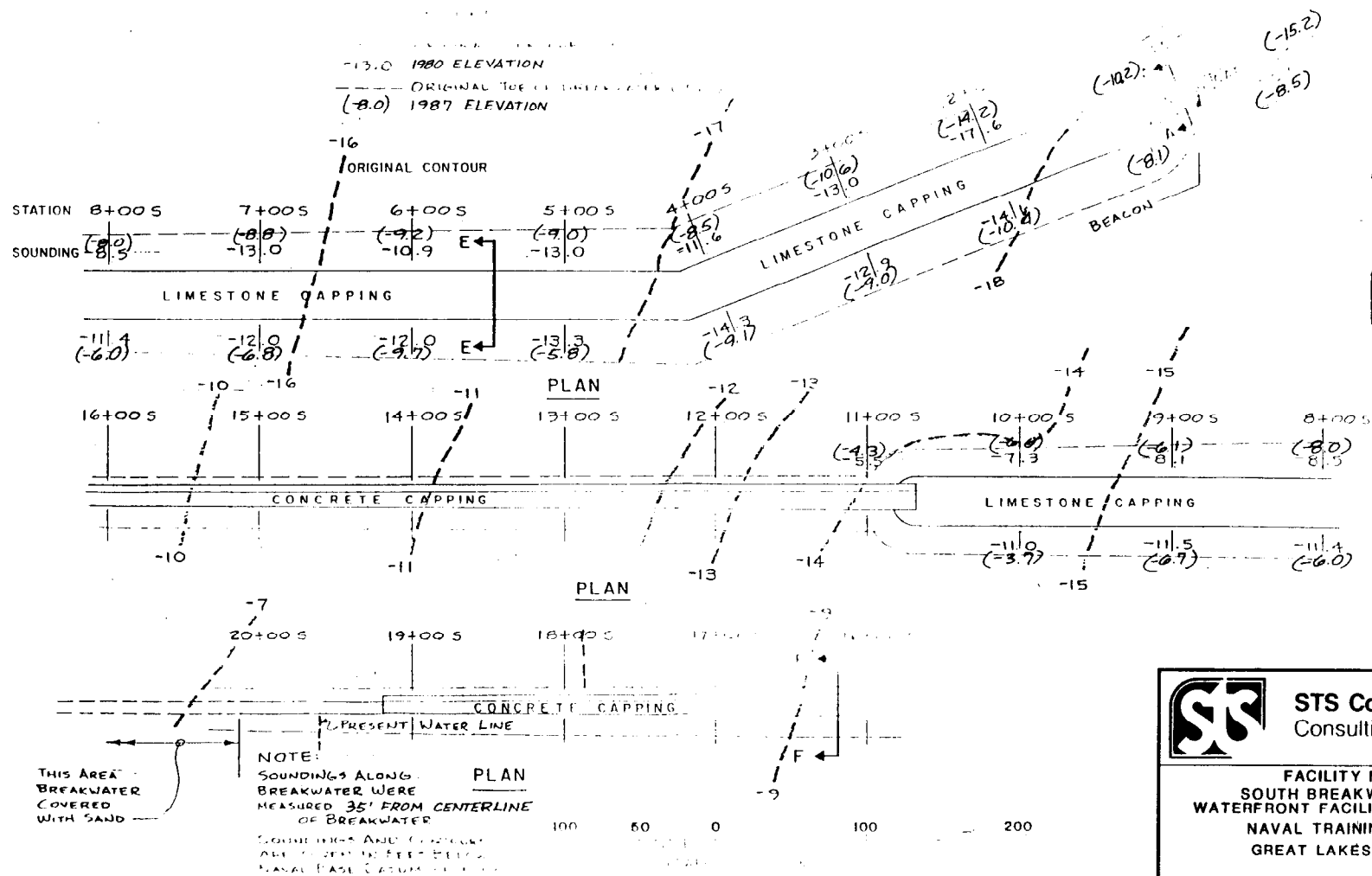
- d) Timber Pile Crib - Although this section has been subjected to some damage, the water depths have been reduced over the years to a point where the structure does not provide critical protection from wave action; however, the integrity of the structure should be maintained. Over time, stones have been removed from underneath the concrete cap due to wave action pulling stones out from between the timber piles. Eventually, it can be expected that the concrete cap will continue to deteriorate and move both horizontally and vertically. To provide protection against failure, the structure should be encased in a stone reventment from Stations 26+20N to 28+20N. The opinion of probable cost for this repair is \$102,000 (\$36,500 material, \$65,500 labor). The probable cost to repair concrete surface deterioration is \$34,000 (\$5,800 material, \$28,200 labor).

4.2 South Breakwater

4.2.1 Description

The south breakwater is located at the southern boundary of the NTC property. The breakwater extends 1497 feet due east, then turns to a bearing of about N 68° E for the remaining 422 feet, as illustrated on Figure 6. The breakwater is comprised of the following three cross section configurations:

- a) Concrete Caisson - The section nearest the mouth of the harbor consists of 108 feet of rock filled reinforced concrete caissons with reinforced concrete capping. Each of the two caissons is 54 feet long. The concrete capping was placed in two sections per caisson. A beacon tower is supported by the 2nd caisson from the harbor mouth. The tower flashes a white harbor light which was designed to be visible for 11 miles. Figure 7 illustrates the cross section of this structure.
- b) Limestone Rubble Mound - The next breakwater section consists of a Bedford limestone rubble mound (station 1+08 to 10+67). The stones were cut to an approximate dimension of five ft by five ft by ten ft. These stones were placed with the long axis parallel to the breakwater as shown in the typical cross section in Figure 8. This portion of the south breakwater is 959 feet long.
- c) Timber Pile Crib - The near-shore end of the breakwater is a rock-filled timber pile crib capped with reinforced concrete. The timber crib was formed by driving two rows of close piles and connecting them by tie rods every 8 feet. Twelve inch square timber wales were attached to each end of the tie rod on the outside of the row piles. Every 3rd close pile on the south side was extended one foot-three inches and every 4th close pile on the north side was extended two feet. The

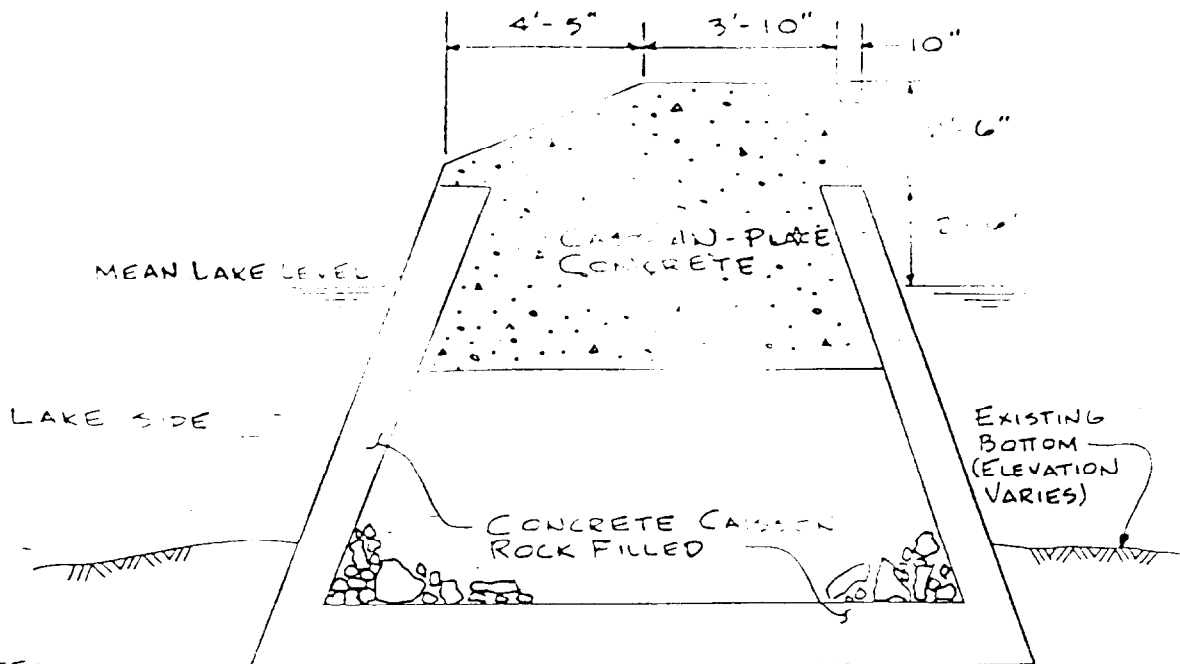


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FACILITY NO. 714
SOUTH BREAKWATER PLAN
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

(REFERENCE 1)

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L.M.B.	6-88	6	

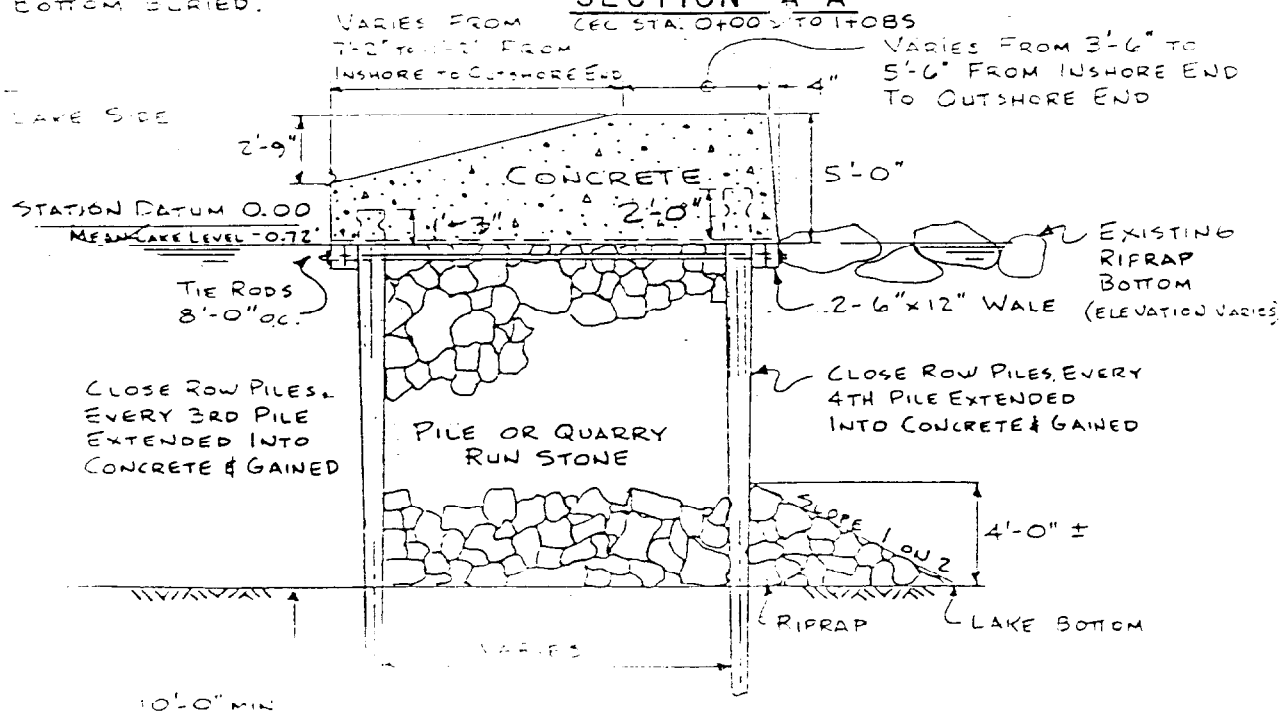


NOTE:

CASSON CONFIGURATION
ASSUMED
BOTTOM BURIED.

FOUNDATION UNKNOWN

SECTION A-A



(CROSS-SECTIONS
OF INSHORE & OUTSHORE
CONCRETE SECTIONS)

SECTION F-F

CEC STA 10+70.5 TO 19+21.5

NOT TO SCALE



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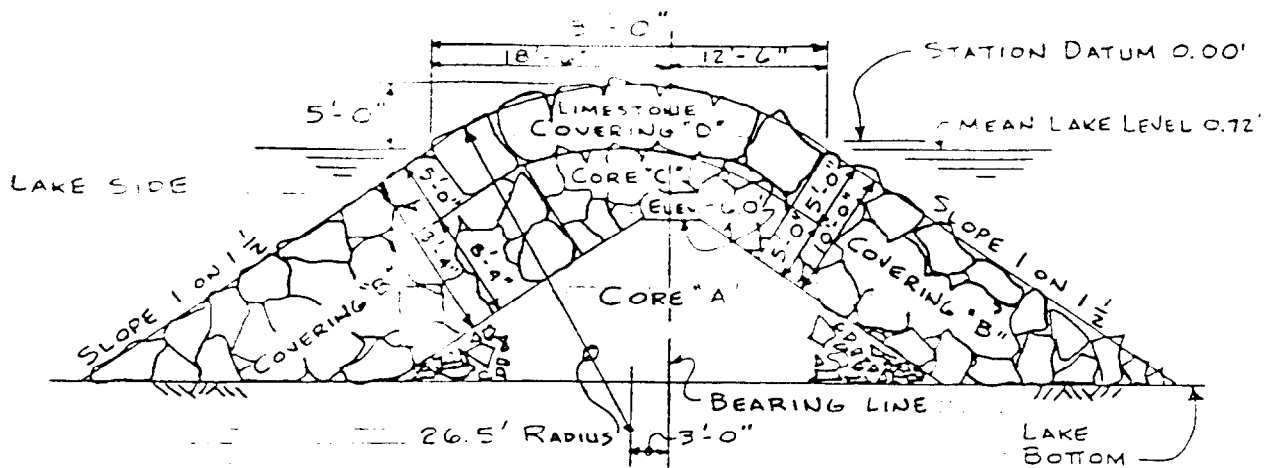
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FIGURE NO

7

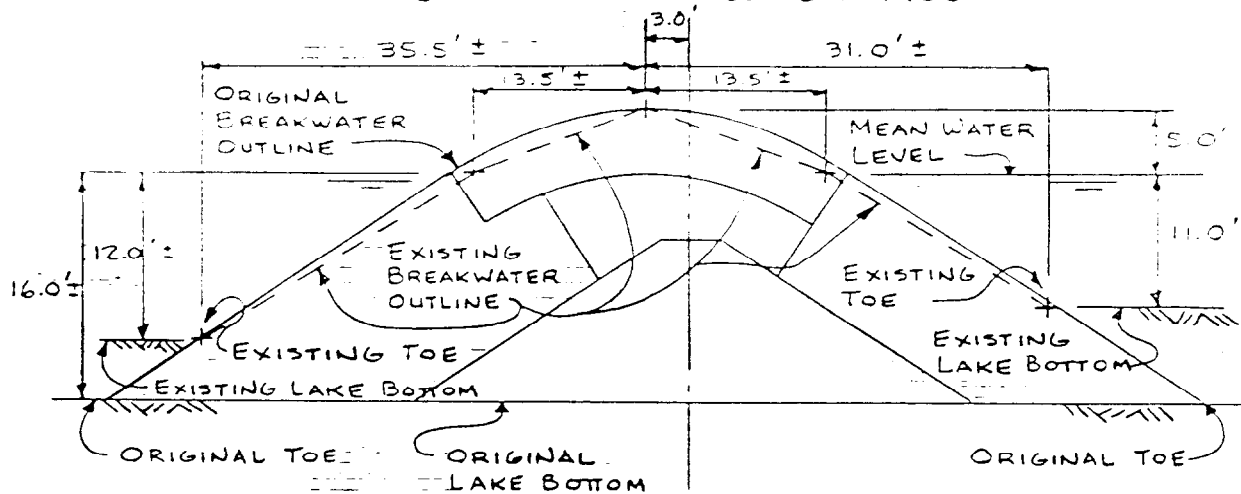
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1137-1



SECTION E-E

ELEVATION OF ORIGINAL DESIGN
CEC STATIONS 1+085 TO 10+705



CROSS-SECTION THROUGH CEC STA 6+005
SHOWING EXISTING BREAKWATER OUTLINE
SUPERIMPOSED ON ORIGINAL OUTLINE

(CROSS-SECTIONS OF LIMESTONE-
CAPPED PORTION OF THE RUBBLE
MOUND SECTION)

NOT TO SCALE



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SHOWN

FIGURE NO. 8

STS DRAWING NO

1137-1

concrete cap was cast in place around the extended piles and on top of the wale. This section is approximately 852 feet long (station 10+67 to 19+19). A typical cross section is illustrated on Figure 7.

4.2.2 Visual Observations

Following is a description of the condition for each of the three breakwater sections:

- a) Concrete Caisson - The concrete caisson is in relatively good condition considering the structure's age. The lakeward edges of the caisson have experienced some spalling, but not to the extent that the north breakwater caissons have. The concrete is not severely delaminated, and only minor settlement and alignment changes have occurred (Photo 1.03). Approximately 20% of the concrete cap surface area has spalled.

With poor underwater visibility conditions (less than 1 ft), the south breakwater appeared to be in good condition. No damage to the concrete caissons below the water line was observed other than hairline cracks.

- b) Limestone Rubble Mound - The Bedford Limestone section portion of the north breakwater is in better condition than the north breakwater (Photo 1.17). This is as expected since wave activity from the southeast is not as severe and frequent as that which occurs from the northeast. The stone appears to have settled somewhat; however, the change in cross section is not substantial. Individual stones on this structure are broken, often into thirds. These broken stones often settle out, leaving voids the size of the original stones in the breakwater crest. These voids should be monitored periodically. Water depths adjacent to this structure do not appear to pose a serious problem at this time, but have become more shallow (Photo 1.09).



Photo 1.03 - Surface spalling on South Breakwater caisson.



Photo 1.17 - Bedford limestone section on South Breakwater.

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Photo 1.09 - Voids in South Breakwater limestone section due to weathering and settlement of stones.



Photo 1.23 - Failure of capping sections on South Breakwater.

The armor stone appeared fairly regular below the water line with no voids apparent. The toe of structure appears to be uniform as evidenced by the side scan sonar survey.

- c) Timber Pile Crib - The capping section for the first 35 ft adjacent to the limestone (Station Nos. 10+80 to 11+15) has spalled completely away from its southern supporting timber piles, and has collapsed. The second capping section (Station Nos. 11+15 to 11+50) is nearing the same level of damage, and the third section (Station Nos. 11+50 to 11+85) has exposed timber piles (Photo 1.23). The remaining sections have experienced surface spalling, particularly at construction joints.

Below the water lines, some loss of stone from the crib observed on the south side of the collapsed concrete capping (Station 11+00), and again near Station Nos. 11+75 and 13+00. At Station 11+75, a void of 1 to 1.5 feet was encountered under the concrete cap, and stone which was washed out of the cribbing was observed on the lake bottom. No wales could be located, and the tie rods were exposed above water. Missing wales and exposed tie rods were also noticed on the north side of the breakwater between 11+00 and 11+50. For the remainder of the concrete capping, the timber piles were covered with concrete rubble and riprap.

4.2.3 Remedial Measures

The south break water is generally in considerably better shape than the north break water. However, the following repairs are recommended: a) repair concrete surface deterioration on the concrete caisson for approximately 20% of the surface area for a probable cost of \$6,000 (\$1,100 material, \$4,900 labor); b) monitor voids and stability of the limestone rubble mound section periodically; and c) place a stone revetment on the north and south side of the timber pile crib for a distance of 150 feet and 250 feet, respectively. The opinion of probable cost to repair the timber pile crib is

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July 29, 1988

\$65,000 (\$22,500 material, \$42,500 labor). Finally, spalled concrete on the timber pile crib cap, which covers approximately 5% of the cap, should be repaired for a probable cost of \$19,000 (\$3,300 material, \$15,700 labor).

4.3 North Pier

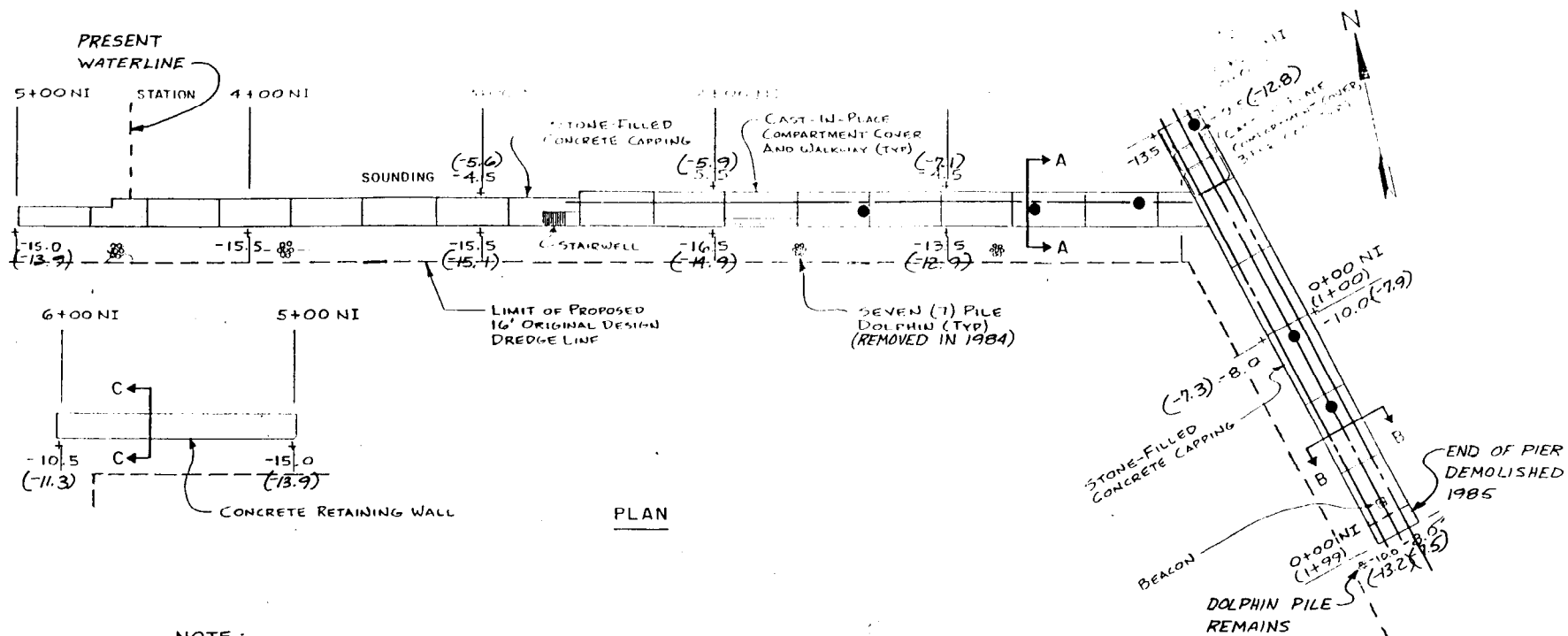
4.3.1 Description

The North Pier was originally constructed as a stone filled timber crib around the year 1919. The structure was replaced by a new structure in 1933. Plan view stationing and details for this pier are illustrated on Figure 9. The pier has three main components: the main pier, the outer leg, and a retaining wall which is located west of the main pier. The retaining wall surrounds the inner harbor and extends to the mouth of the boat basin. Following is a description for each pier component.

- a) Main Pier - The main pier is 504 feet long and lies on a bearing of S 78° E. The pier is a stone-filled crib of precast reinforced concrete constructed over the original timber crib base. The base consists of close driven round piles on the north side and timber sheet piles and fender piles on the south side. Fender wales were added on the south side of the pier. Each concrete crib section is typically 31 feet long and six feet tall, and has two cross beams which divide the section into thirds. A typical cross section is illustrated on Figure 10.

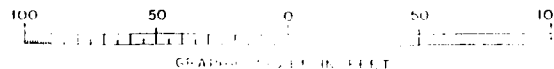
Stationing was established beginning at the east end of the main pier and increasing westward. The letters NI are added to the station to designate the North Pier. The main pier is comprised of three cross sections which are similar except for the width of the concrete crib.

The outermost section of the main pier, from station 0+00 NI to 2+64 NI, gradually narrows in width from 14.5 to 14.0 feet. The center section, from 2+64 NI to 4+63 NI, is 11.5 feet wide. The nearshore section, from station 4+63 NI to 5+04 NI, is 8.0 feet wide. These widths do not include timber wales which extend an additional six inches on each side of the pier. The nearshore section and most of the center section are landfilled on the north side.



NOTE :
SOUNDINGS ARE GIVEN
IN FEET BELOW NAVAL
BASE DATUM OF 0.00'.

NON-DESTRUCTIVE TESTING LOCATION
SIR SURVEY
CONCRETE CORE

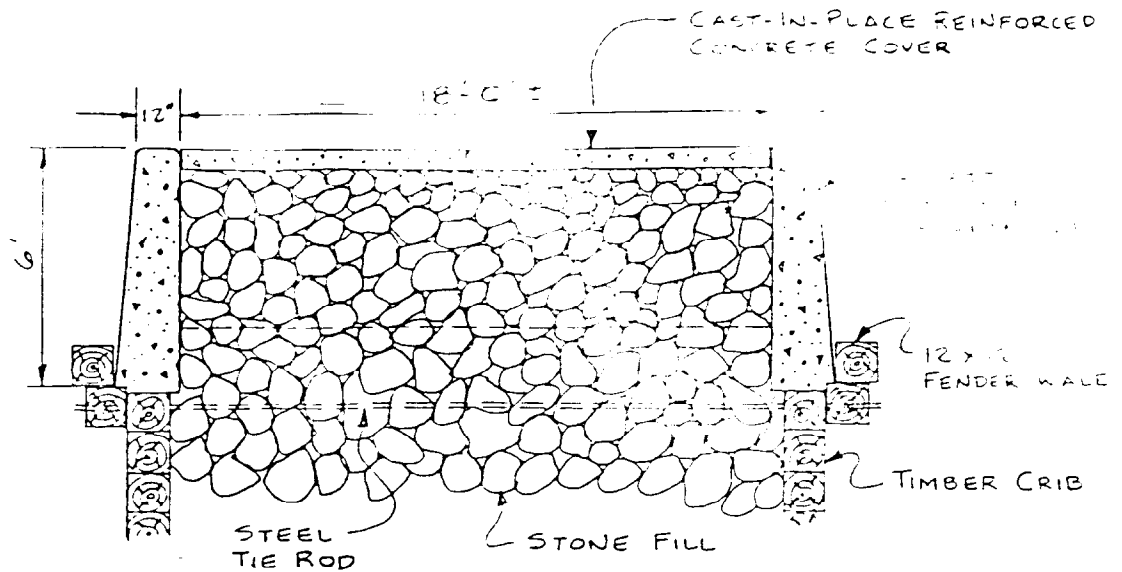


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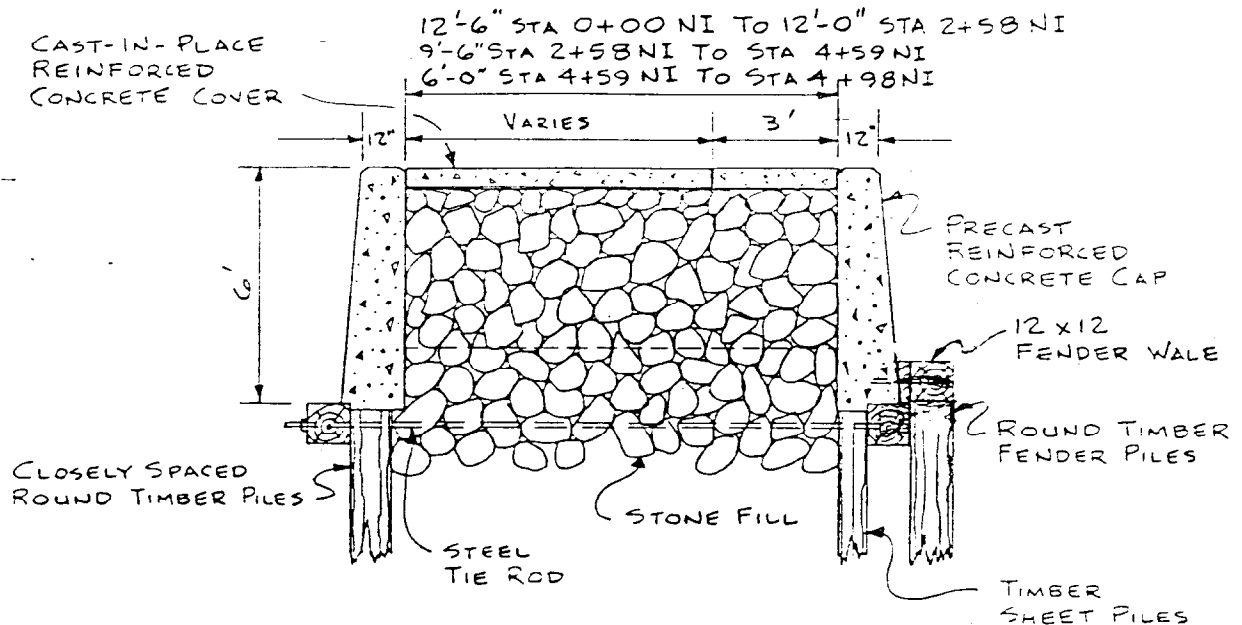
**FACILITY NO. 717
NORTH PIER PLAN
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS**

(REFERENCE 1)

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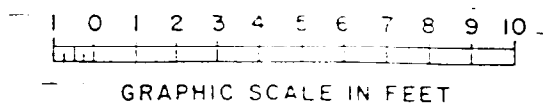
SECTION B-B.



SECTION A-A

CEC STATION 0+00 NI TO 4+98 NI

(CROSS-SECTIONS OF
LEG AND MAIN PIER)



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PROJECT/CLIENT

FACILITY NO. 717
NORTH PIER SECTIONS
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

DRAWN BY G.R.S. 5-88

CHECKED BY L.M.B. 6-88

APPROVED BY

SCALE SHOWN FIGURE NO 10

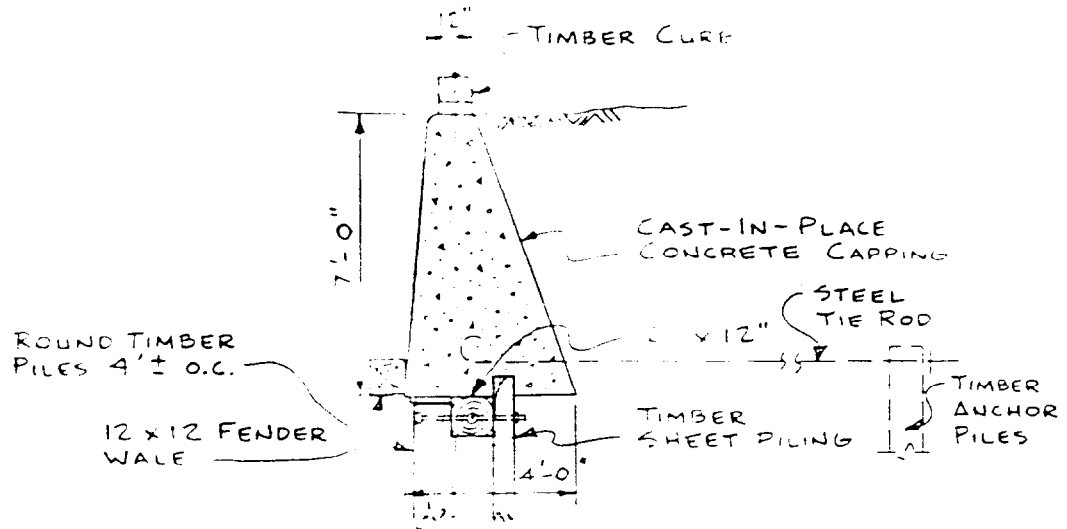
STS DRAWING NO. 1137-1

A concrete walk three feet wide was cast over the stone fill adjacent to the south edge of the North Pier during the 1933 construction. The remainder of the pier was covered at a later date. A small boat landing was constructed on the center section near station 2+64 NI.

- b) Outer Leg - The outer leg of the North Pier is 197 feet long and lies on a S 16° E bearing. The outer section was originally a timber crib which was capped with a precast reinforced concrete crib sometime after 1933. Stationing on the outer leg was established from north to south. The stationing is indicated by 0+00 NI followed by the leg station in parentheses.

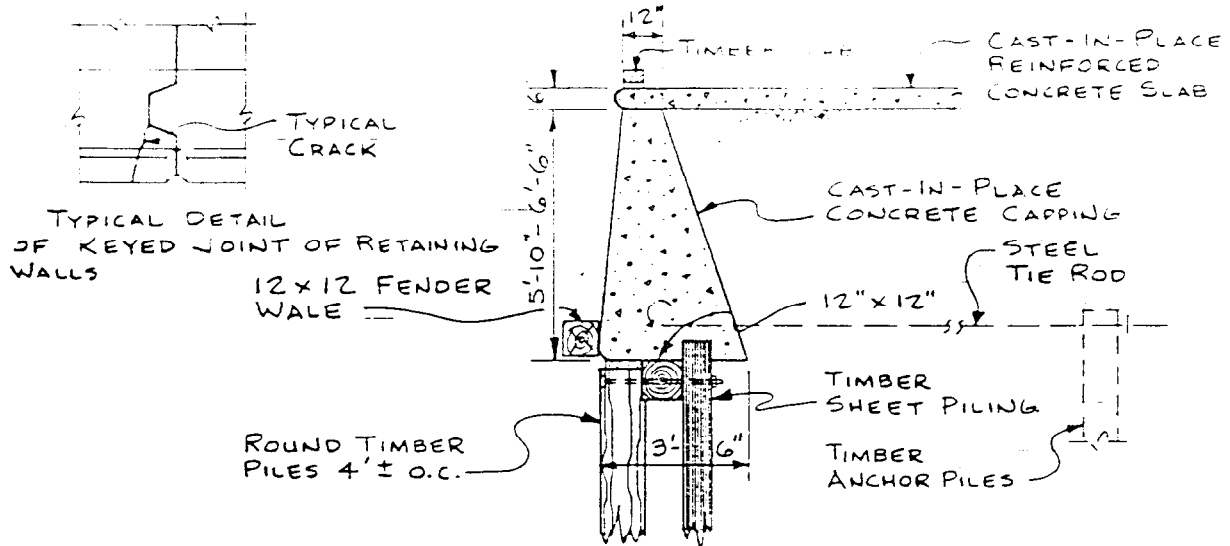
The timber crib consists of interlocked horizontal beams twelve inches square. The concrete cribbing is similar to the main pier, except each section is about 33 feet long and 20 feet wide. The stone fill in the cribbing was also capped at a later date. Unlike the main pier, which was capped with a continuous slab, the outer leg was capped by casting individual slabs over each compartment. A typical cross section is shown on Figure 10.

- c) Retaining Wall - The inner harbor wall between the North Pier and the Boat Basin is supported by a retaining wall. There are two typical cross sections for this area. The north wall, from station 5+04 NI to 6+14 NI, originally consisted of a timber pile crib revetment. In 1933, the piling was cut off and capped by a reinforced concrete retaining wall. The original pile tiebacks were incorporated into the concrete structure. The area behind the retaining wall has been resurfaced several times over the years, and currently serves as a parking lot. A typical cross section is presented on Figure 11.



SECTION C-C'

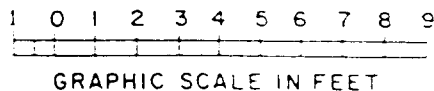
SOUTH PIER RETAINING WALL
CEC STATION 4+35 SI TO 7+19 SI



SECTION C-C

NORTH PIER RETAINING WALL
CEC STATION 4+98 NI TO 6+00 NI

(CROSS-SECTIONS OF
RETAINING WALLS)



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PROJECT/CLIENT

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NORTH PIER SECTIONS
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NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

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SCALE
SHOWN

FIGURE NO
11

STS DRAWING NO

1137-1

The west harbor wall was originally a wooden and steel sheet pile seawall backed by a concrete retaining wall. In 1973, a new steel sheet pile wall was driven about one foot lakeward of the old wall. The new sheet piles are tied to anchor piles located 20 feet west of the seawall. The tierods are spaced at 6 to 8 foot intervals. A concrete cap was poured between the steel sheet pile wall and the original concrete retaining wall, the top two feet of which are still visible. This portion of the retaining wall was restationed beginning at the north end and increasing to the south. The station is designated by 6+00 NI followed by the wall station in parentheses.

4.3.2 Facility Observations

Following is a condition assessment for each component of the north pier:

- a) Main Pier - The north pier has experienced minor surface spalling between the water line and the top of the structure (Photo 7.17). Spalling is typically less than 2 inches deep, and occurs at construction joints and at the top edge of the structure. Patching of spalled areas has been performed in the past. In many cases, the patches do not appear to have bonded well to the existing concrete, and are being eroded away. The spalling should be monitored and patched when the reinforcing steel is in danger of becoming exposed.

Extensive concrete loss has occurred at construction joints at Stations 3+28, 3+90, and 4+15. At these locations, the entire thickness of the concrete capping wall appears to have been lost. Previous attempts at patching are apparent; however, the attempts focused on the top of the section and holes were left in the vertical face of the pier.



Photo 7.17 - Surface spalling on North Pier.



Photo 8.01 - Collapsed section of North Pier.

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Photo 7.15 - Settlement cracking on North Pier.

2 141111 97 550

From Station 0+60NI to 2+50NI, small lengths of the upper wale were found to be missing. Another loose tie bolt was observed at 0+90NI. Damaged tie rods were noticed at Station 5+50.

The underwater investigation reveals that the timber piles and timber cribbing are in good condition with no signs of deterioration or decay. Some stone leakage was noticed on the north side near Station 1+75 and from Station 2+75 to 4+00; however, the spillage was described as minor. At Station 2+45, chains have been attached to the tie rods, probably as a docking aid. One of the tie rod bolts was loose and could be manipulated by hand.

- b) Outer Leg - The southern 1/3 of the southern outer leg capping section has collapsed. (Photo 8.01) It appears that the capping section may have been struck by a ship attempting to dock. The timber crib is still intact. NTC drawings indicate that this section has been damaged and replaced in the past.

The slabs over the capping compartments on the outer leg appear to have settled on the order of 1 inch at the edges, resulting in cracks along the centerline of the slabs (Photo 7.15). The possibility exists that voids under these slabs are due to settlement of the supporting stone fill. On the south side of the pier, a section of missing wale was identified from Station 0+00NI (0+75) to 0+00NI (1+00). An exposed tie rod with a missing nut was also observed.

Underwater observations indicated evidence of the presence of small stone at the base of the outer leg cribbing. It appeared that this material was placed for protection and did not originate from within the cribbing. No cribbing holes were observed. The south end of the outer leg cribbing appears to be undamaged; however, the concrete capping has been destroyed.

- c) Retaining Wall - The retaining wall section is in good condition, with some surface cracking and minor spalling. The concrete quality is generally good, with small areas of severe delamination located at the north end of the outer leg, and near the small boat landing. No major alignment or settlement problems were observed.

4.3.3. Non-Destructive Tests Results

A subsurface interface radar survey was performed on top of the concrete surfaces of the North Pier. The survey extended above the main section of the pier and the outer leg. The objective of the survey was to identify the presence of the voids under the concrete slab that could impact the structural ability of the slab to support loads. Results of the survey indicated the potential presence of minor voids in areas; however, no major voids were indicated. Numerous concrete cores were performed after the SIR Survey to investigate the potential void areas. Three cores were performed on the outer leg. No voids or free space were observed beneath the slab at all three locations. The cores ranged in thickness from five to seven inches, and Core No. 1 encountered reinforcing bars near the base of the slab. Loose gravel, angular limestone and concrete cobbles were noted in Core No. 1. Loose gravel and black, asphaltic gravel were noted in Core Nos. 2 and 3. No air pressure differences due to wave swell were noted below the slab.

Three concrete cores were performed on the main pier. Small voids, on the order of one to one and one half inches were observed beneath the slab at Core Nos. 4 and 6. Core No. 5 did not indicate any free space below the slab. Gravel, concrete cobbles and asphaltic gravel were observed at coring location No. 4. Loose gravel was observed in Core Nos. 5 and 6. Two reinforcing bars were encountered in Core Nos. 5 and 6 while three levels of bars were found in Core No. 4. Changing air pressure from wave swells was noted at all three core locations.

It appears that the minor voids encountered under the slab are not extensive enough to have an impact upon the structural ability of the piers to withstand loads.

4.3.4. Structural Analysis

Structural analysis was performed for the North Pier to estimate the maximum allowable live and vehicular loadings. An attempt was made to accommodate the current condition of the pier in the analysis to develop maximum loading conditions.

For purposes of analysis, the stone and gravel fill within the piers was assumed to have a unit weight of 130 lbs per cu ft and an angle of internal friction of 35° . These values are typical for large granular soil material. The reinforcing steel was assumed to have a yield strength of 20,000 lbs per sq inch. The compartment walls located at the 1/3 points of each capping section act as lateral supports. A typical section was evaluated consisting of a single-side wall with ten feet length. This section was analyzed for active failure from within the structure with a surcharge acting on the compartment cover slab. Analysis results indicate that an average surcharge of 400 lbs per sq ft should be allowable on the pier surface.

Structural analyses for the north retaining wall assumed that the soil behind the wall has a unit weight of 125 lbs per cu ft, and an angle of internal friction of 30° . These values are representative of sand and cohesionless material. The tie rod was assumed to have a yield strength of 20,000 lbs per sq in.

The retaining wall was analyzed for an overturning failure mode. The deep piles that are present indicate that this mode of failure would be more likely than sliding. The analysis included passive and active earth pressures, dead weight of the concrete wall, and the restraining force of the tieback system. Results of the analysis indicate that the wall should remain stable under a surcharge loading of 300 lbs per sq ft. Thus,

live loading and vehicles with a weight distribution of up to 300 lbs per sq ft may be placed adjacent to the retaining wall. Please note that this analysis assumes that the condition of the tieback system is acceptable and uniform.

4.3.5. Remedial Measures

The below water section of the entire North Pier appears to be in reasonable condition and is not in need of any repair at this time. However, several areas above the waterline will require some attention. The most serious problem exists at the southern edge of the outer leg. Failure of the structure was noted in the 1980 Inspection Report. This failure has continued to a point where the concrete cap and is no longer present. The timber crib below the waterline appears to be in good condition and the stone fill terminates at the top of the timber. Portions of the concrete slab which have now failed rest on top of this section. This structure should be repaired to maintain the integrity of the remainder of the outer leg. Furthermore, some means of warning boats of the underwater hazard should be considered. The compartment that has failed could either be removed and rebuilt, or the entire end of the pier could be buried in a stone revetment. Replacing the existing structure would involve the removal of all rubble from around the timber piers and from above the upper limit of the timber crib. A steel sheet pile wall could then be driven around the perimeter of the failed cell and extend up above the waterline. The structure could then be filled with stone and capped with concrete. The opinion of probable cost to accomplish this repair would be approximately \$87,000 (\$45,300 material, \$41,700 labor).

The remainder of the main pier and the outer leg is in need of repairs to the spalled concrete surface. Spalling has occurred on approximately 10% of the surface to a point where structural integrity could be in question. These surfaces should be repaired by the removal of poor concrete and replacement with new concrete. Attention should be paid to areas previously repaired where gaps exist at the waterline. These locations

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July 29, 1988

are at Station Nos. 3+28, 3+90, and 4+15. The estimated opinion of probable cost to accomplish repairs to the spalled areas is estimated to be \$40,000 (\$6,900 material, \$33,100 labor).

The north retaining wall showed no unusual conditions. Remedial measures are not necessary at this time.

Note that the 1980 Inspection Report indicated that the water depth along the south side of the pier is deeper than the designed dredged depth by up to six feet. Although this has not been a problem in the past, it was recommended that small-diameter stone should be placed at the base of the timber foundation to alleviate excessive stresses. Water depths do not appear to have been addressed in the past and this recommendation should still be in effect. The opinion of probable cost to accomplish this work is of \$16,000 (\$11,600 material, \$4,400 labor).

The remaining useful life of the entire structure with the recommended repairs in-place should be 10 - 15 years. With a routine maintenance and repair program, this useful life could be extended.

4.4 South Pier

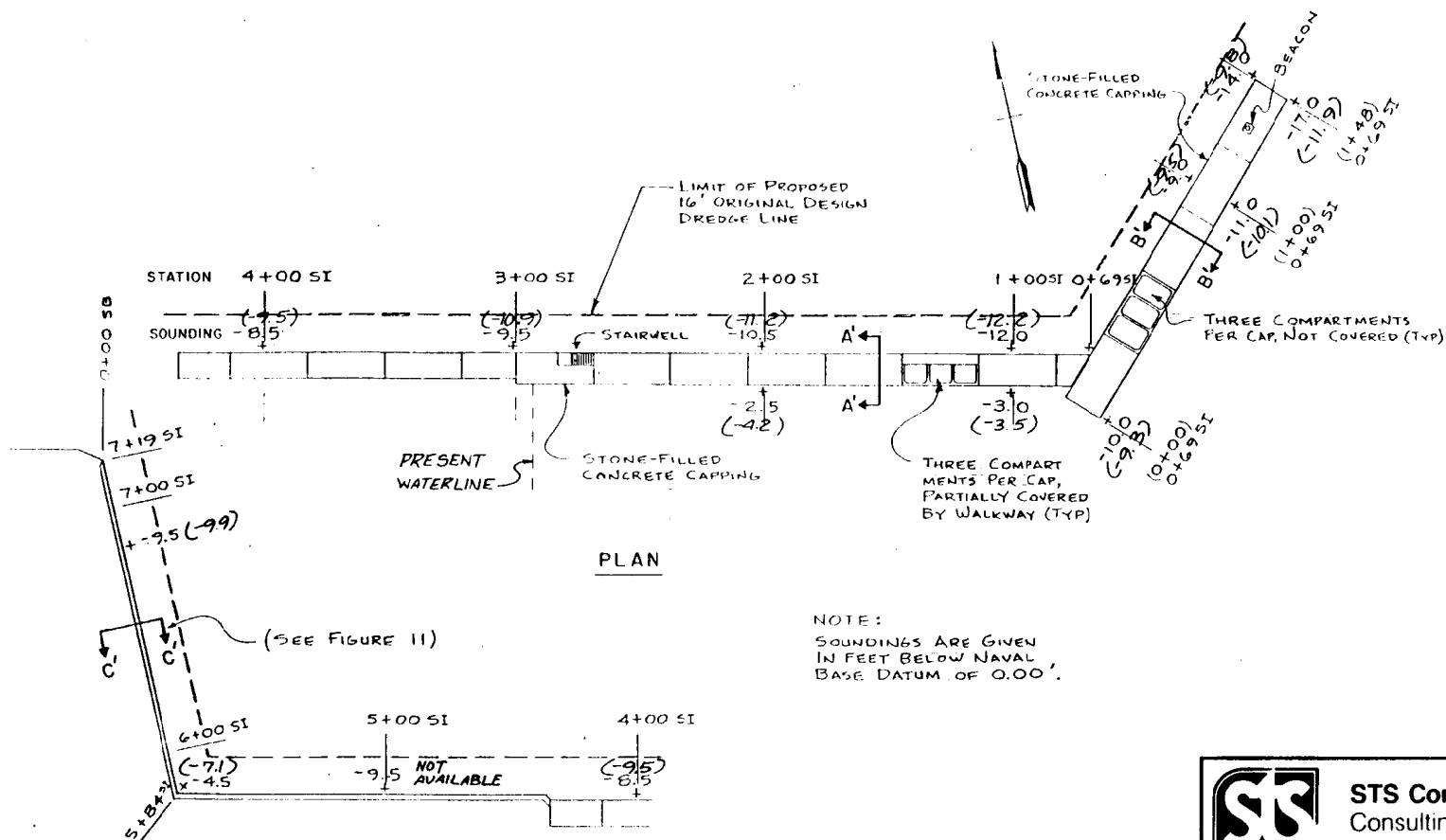
4.4.1. Description

The original South Pier was a stone filled timber crib constructed in 1919 and replaced in 1933 by the present structure. A plan view of the stationing and configuration of the south pier is illustrated on Figure 12. The pier has three main components: the main pier, the outer leg, and the onshore retaining wall. Following is a description of each pier component:

- a) Main Pier - The main pier is 360 feet long and is parallel to the North Pier on a bearing of S 78 E. The pier is a stone-filled crib of precast reinforced concrete constructed over the original timber crib base. The base consists of close driven round piles on the south side and timber sheet piles and fender piles on the north side. Fender wales were added on the north side of the pier, but were removed in the mid 1970's when the Synchronlift was constructed. Each concrete crib section is typically 31 feet long and 6 feet tall, and has two cross beams which divide the section into thirds. A typical cross section is illustrated on Figure 13.

Stationing was established, beginning with 0+69 SI, at the east end of the main pier and increasing westward. The stationing was offset in this way to be symmetric with the North Pier stationing. The letters SI after the station are used to designate the South Pier. The main pier is comprised of two cross sections which are similar except for the width of the concrete crib.

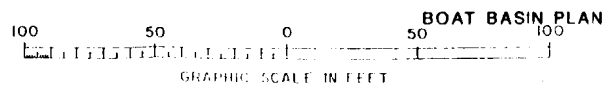
The outermost section of the main pier, from station 0+69 SI to 2+94 SI, is 12.25 feet wide. The inner section, from 2+94 SI to 4+29 SI, is 10.25 feet wide. These widths do not include timber wales which extend an additional six inches on each side of the pier.



PLAN

NOTE:
SOUNDINGS ARE GIVEN
IN FEET BELOW NAVAL
BASE DATUM OF 0.00'.

PLAN

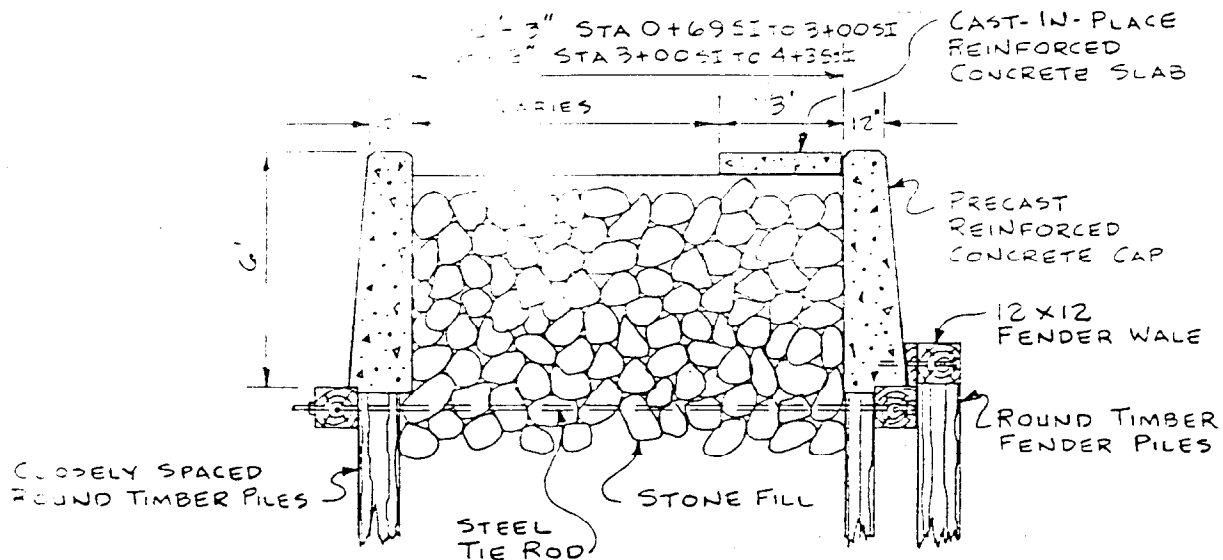
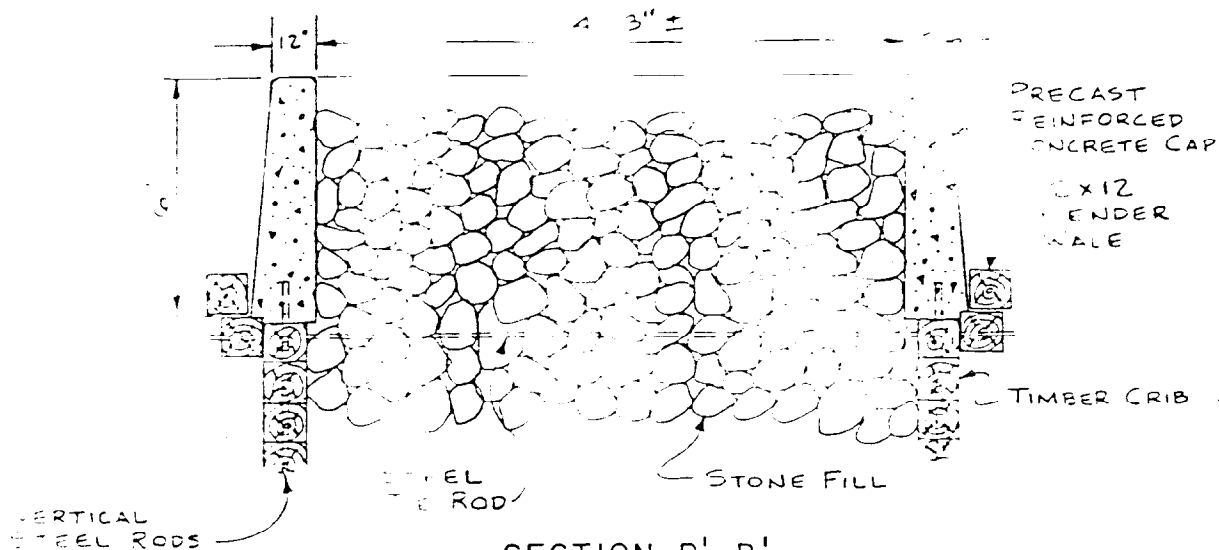


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SOUTH PIER PLAN
WATERFRONT FACILITIES INSPECTION
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GREAT LAKES, ILLINOIS

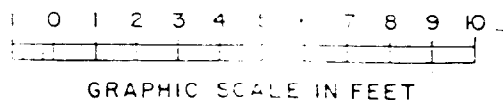
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CHECKED BY	DATE	SHEET NO.	SIS FILE NO.
L.M.B.	6-88	12	



CEC STA 0+69.51 TO 4+35.51

CROSS-SECTION OF LEG AND MAIN PIER



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FACILITY NO. 716
SOUTH PIER SECTIONS
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L.M.B.

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SCALE

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FIGURE NO

13

STS DRAWING NO.

1137-1

A concrete walk three feet wide was cast over the stone fill adjacent to the north edge of the South Pier. Small concrete slabs were later cast to install bollards, but most of the concrete cribbing remains uncapped. A small boat landing was constructed on the center section near station 2+71 SI.

- b) Outer Leg - This portion of the South Pier is 148 feet long and lies on a N 42° E bearing. The outer section was originally a timber crib which was capped with a precast reinforced concrete crib sometime after 1933. Stationing on the outer leg begins at the south end and increases northward. Stationing is designated by 0+69 SI followed by the leg station in parentheses.

The timber crib consists of interlocked horizontal beams twelve feet square. The concrete cribbing is similar to the main pier, except each section is about 30 feet long and 16 feet wide. The concrete cribbing compartments on the South Pier are uncapped. A typical cross section is shown on Figure 13.

- c) Retaining Wall - The inner harbor wall between the South Pier and the Boat Basin is supported by a retaining wall. The south wall, from station 4+29 SI to 5+84 SI, and the west wall, from station 5+84 SI to 7+18 SI, were constructed in 1933, and have the same configuration. The original wall consisted of a timber sheet pile seawall with fender piles every four feet and tiebacks every eight feet. In 1933 the piling was cut off and capped by a reinforced concrete retaining wall. The original pile tiebacks were incorporated into the concrete structure. The area behind the retaining wall is grassed and is not subject to heavy loading. A typical cross section is presented on Figure 11.

4.4.2. Visual Observations

Following is a description of the structural conditions of each pier component:

- a) Outer Leg - The outer leg of the south pier has experienced spalling similar to the north pier. The spalling occurs near and above the water line on the vertical face of the leg, and at the upper edge of the concrete capping. The spalling is generally minor, except for the northwest and northeast corners (Stations 0+69SI(0+00)), where up to two feet of concrete has been lost. The remaining concrete is severely delaminated in these areas (Photo 7.22). Concrete spalling on the entire south pier affects approximately 5% of the surface area.

A section of the concrete capping has been damaged on the west side of the outer leg. It appears that the capping was struck by a boat, removing a section of concrete which is still attached by its reinforcing steel. This damage was noticed in the 1980 inspection report and has not been addressed.

The outer leg compartments are filled with approximately eight inch diameter stone, which appears to have settled to just above the harbor water level. The main leg compartments have been filled with a similar stone, although they appear to have been backfilled more recently with crushed stone or gravel to a level two feet below the top of the compartment.

- b) Main Pier - Following are locations of notable damage along the main pier: Station 3+80 - spalling at construction joint, exposed rebar. Station 5+10 - spalling at construction joint, metal ring support exposed (Photo 6.22). Station 4+12 to 4+22, cracking and damage to vertical pier face which may be collision damage.



Photo 7.22 - Spalling of outer corners on South Pier.



Photo 6.22 - Severe spalling on South Pier.

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In general, the south pier appears to be in better condition than the north, with less spalling and cracking of concrete.

- c) Retaining Wall - The retaining wall sections of the pier structure are generally in good condition. Some settlement of the earth behind the retaining wall has occurred, specifically at stations 5+95 to 6+03 and 6+40 to 6+55. A few of the timber curb sections are missing, badly decayed, or splintered and should be replaced. The damage to the construction joint at station 7+03 noted in a previous report still exists.
- d) Underwater Inspection of Timber Cribbing - As on the north pier, the timber piles and cribbing are in good condition, with no signs of decay or deterioration. The timber crib structure appears to have been undercut at the northwest corner of the outer leg. This may have occurred due to strong eddy currents at this location. The damage at this area has been videotaped. Stone has been placed uniformly at the cribbing base to provide protection. The wales on the north side of the south pier have been removed westward of station 3+00.

Timber below the waterline exhibited softness on the outer 1/2 to 1 inch of the piles. The timber pile heads were soft and split down to the tierods somewhat to that which was noted in the 1980 Inspection Report. This softness is typical of many of the piles along the NTC waterfront and is not expected to cause a structural problem at this time.

4.4.3. Structural Analysis

The structural analysis of the South Pier is similar to that of the North Pier as described in Section 4.3.3. This analysis indicated that an average surcharge of 400 lbs per sq ft should be allowable on the pier's surface. The retaining walls should

be able to withstand a surcharge loading of 300 lbs per sq ft. Since there is no pavement adjacent to the retaining wall which would distribute the load more efficiently, it is recommended that vehicular loads be kept away from the wall. This should not be a problem since there are no parking areas adjacent to the wall.

4.4.4. Remedial Repairs

In general, the South Pier is in good condition. This structure is expected to have a remaining useful life of between 10 and 15 years. This period could be extended with the implementation of a regular maintenance program. Approximately 5 - 10% of the concrete surfaces along the South Pier area have experienced spalling to a depth of three or more inches from the original surface. These areas should be rebuilt. The opinion of probable cost to accomplish this is \$26,000 (\$4,500 material, \$21,500 labor).

Other repairs which could be considered for a nominal cost would be the replacement of deteriorated sections of timber curbing along the retaining wall.

4.5 Boat Basin Seawall

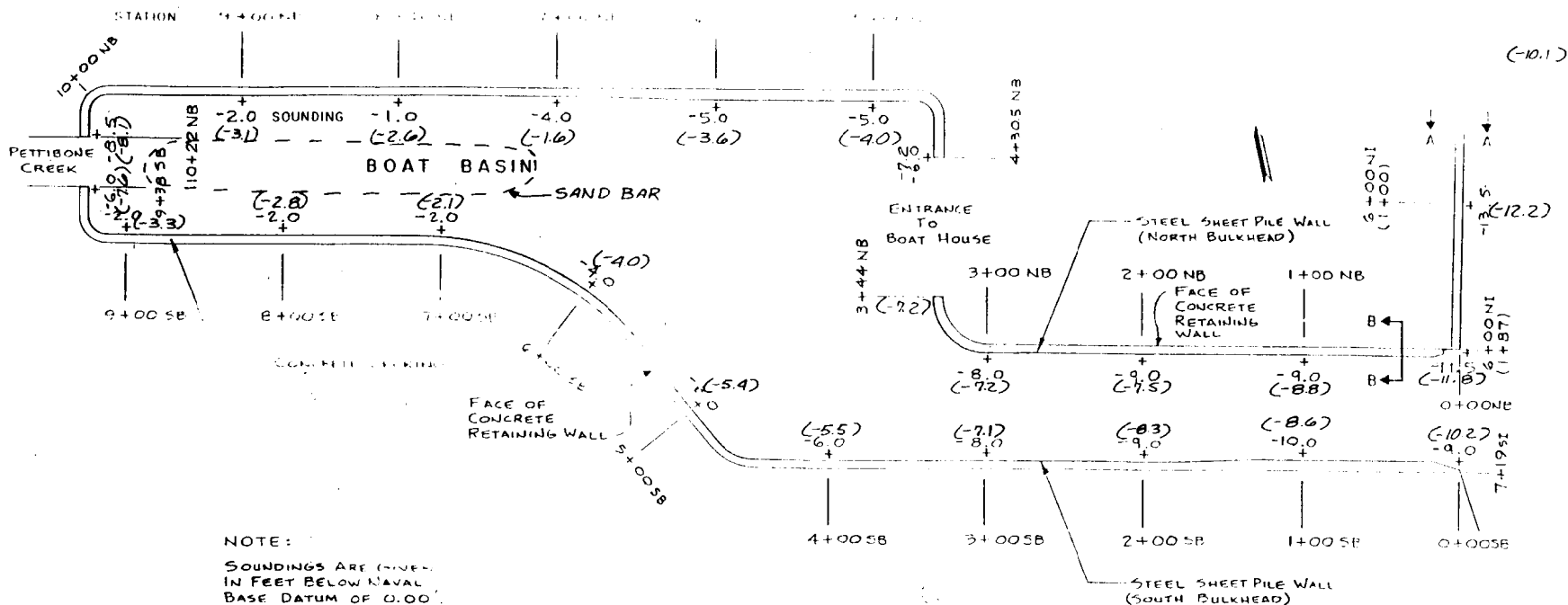
4.5.1. Description

The boat basin is located at the mouth of Pettibone Creek. The basin was originally constructed in 1909 and consisted of a concrete retaining wall supported on timber sheet piles and batter piles, and tied back to anchor piles at eight to ten foot intervals. A wooden railing and fender were attached to the concrete wall. A second tieback system was added in 1944. Figure 14 illustrates the plan view configuration of this basin.

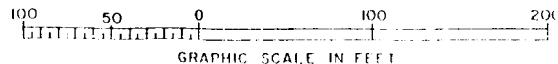
In 1953 the basin wall was reconstructed by installing steel sheet piling five to 6.5 feet in front of the concrete wall. The top of the sheet pile is two feet lower than the top of concrete. A concrete slab five feet thick was placed over fill between the two walls. The new structure was tied to the old concrete wall at about ten foot intervals (Figure 15).

The Boat House (Building 13) is located adjacent to the north wall from station 1+25 NB to 4+30 NB. The access road to the southern shoreline lies adjacent to the remainder of the basin. A narrow bridge over the mouth of Pettibone Creek is located at the basin's western end. The roadway consists of a bituminous paved surface from the Boat House to the bridge. The road south of the basin is unpaved.

Stationing for the Boat Basin begins at the inner harbor and increases to the Pettibone Creek bridge at the west. The north and south walls were stationed independently; the letters SB and NB designate South Basin wall stations and North Basin wall stations, respectively. The north wall is stationed from 0+00 NB at the inner harbor to 10+40 NB at the Pettibone Creek Bridge. The south wall is stationed from 0+00 SB at the inner harbor to 9+45 SB at the bridge.



NOTE:
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BASE DATUM OF 0.00.

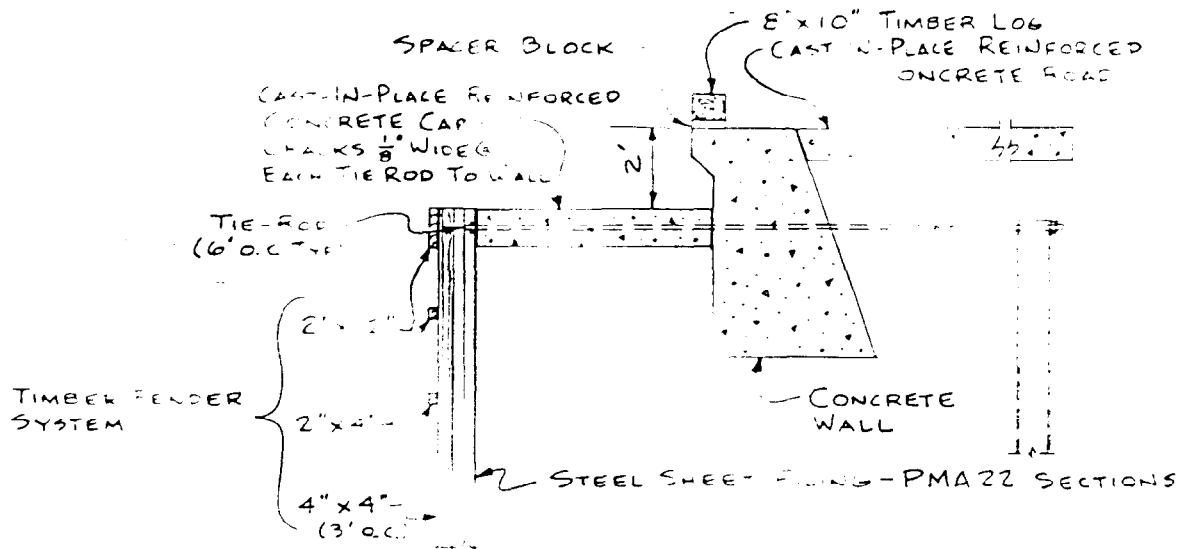


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FACILITY NO. 718-720
BOAT BASIN PLAN
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

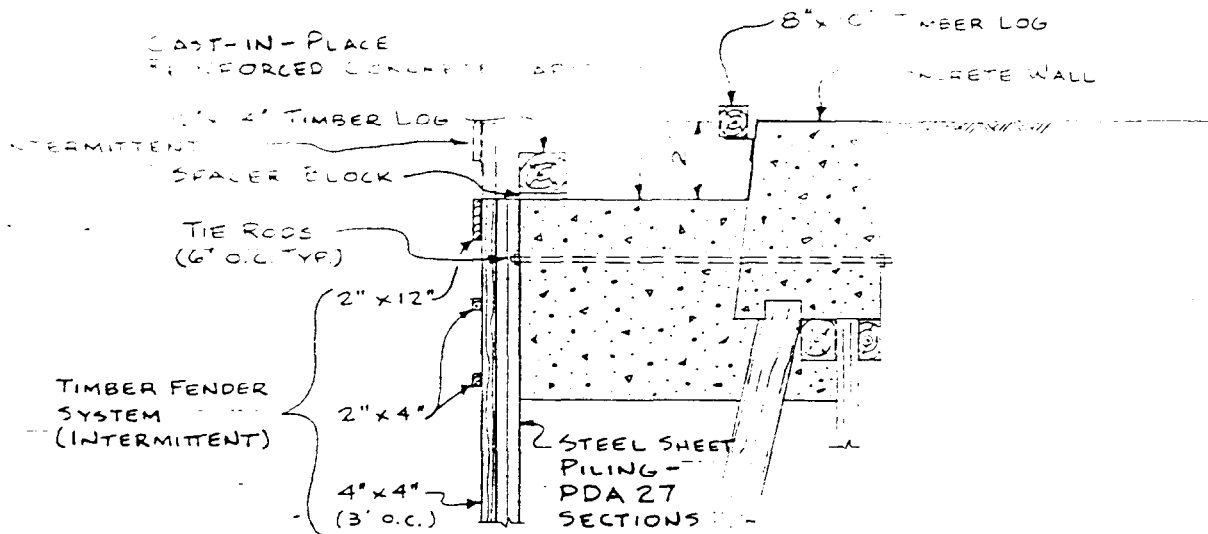
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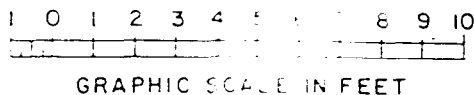
SECTION A-A

CEC STATIONS 6+00 NI (0+00) TO 6+00 NI (1+87)



SECTION B-B

CEC STATIONS 0+00 NB TO 3+44 NB, 4+30.5 NB TO 10+22 NB
CEC STA. 0+00 SB TO 9+38 SB



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FACILITY NO. 718-720
BOAT BASIN SECTIONS
WATERFRONT FACILITIES INSPECTION
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GREAT LAKES, ILLINOIS
(REFERENCE 1)

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SCALE SHOWN FIGURE NO. 15

STS DRAWING NO. 1137-1

4.5.2. Visual Observations

The sheet pile basin wall appears to be in excellent condition. No signs of deterioration of the steel were observed, and all sheet pile joints appeared intact. Extensive steel thickness testing was performed for this wall in 1980. It was reported the steel thickness remained close to the 1953 installation condition. Visual observation of the wall confirmed that no additional thickness testing is necessary at this time.

The concrete walkway is in fair condition, with spalling and missing concrete occurring at the west one third of the basin. The surface spalling is minor in nature, but the areas where concrete has been lost adjacent to the sheet pile section should be repaired. The damage may be due to standing water which ponds on the walkway. If this slab were recast, it should be pitched to drain into the boat basin (Photo 10.22).

The original concrete retaining wall located behind the sheet piling and walkway is badly spalled and eroded. This damage appears over more than 50% of the south basin perimeter. A concrete cap was cast on top of this wall to mount a small guardrail. The spalling of the original concrete has undermined the support for the cap and guardrail, resulting in unstable and potentially unsafe conditions.

Similar instability was observed behind the north basin wall. On this side of the basin, an asphalt roadway is located immediately adjacent to the original retaining wall. Damage and loss of the original retaining wall has resulted in erosion and failure of the roadway embankment at the following stations 7+52NB, 7+70NB, 7+80NB, 8+50NB, 8+85MB, 9+50NB, 9+75NB, and 10+04NB. This problem may be exacerbated by stormwater runoff at the edge of the roadway (Photo 11.02).



Photo 10.22 - Poor drainage on Boat Basin deck.

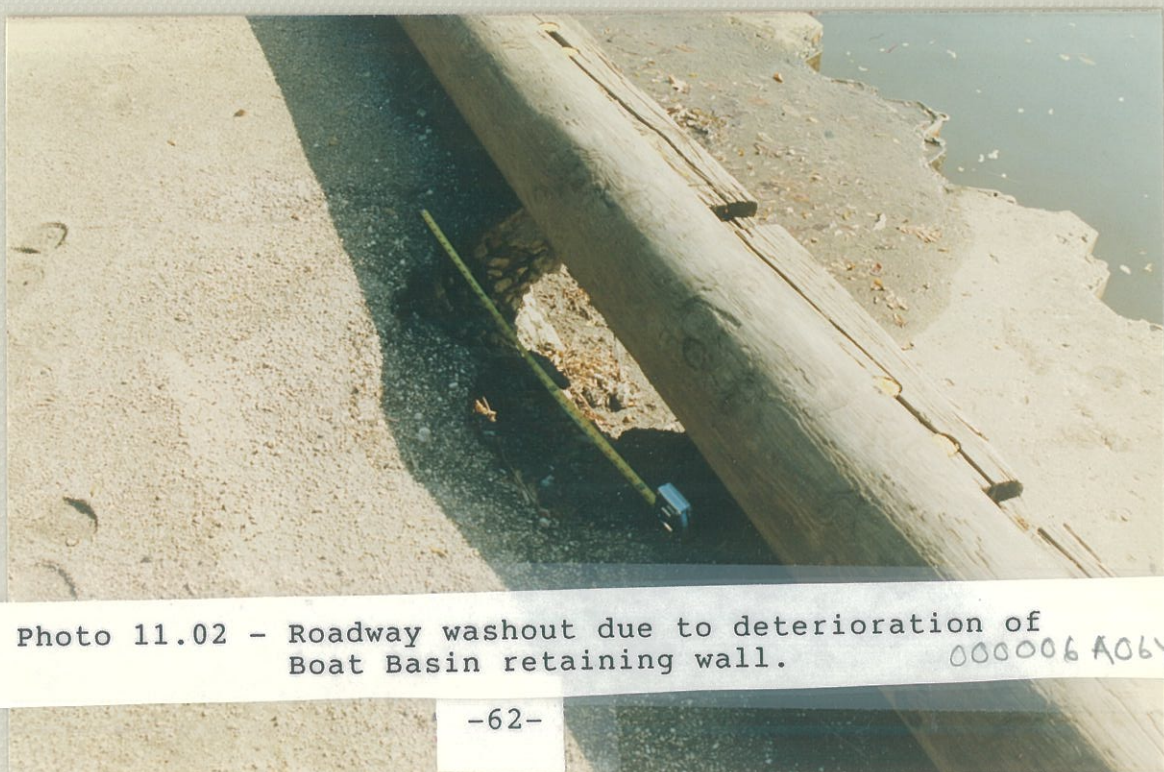


Photo 11.02 - Roadway washout due to deterioration of Boat Basin retaining wall.

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The basin sheet pile wall is in excellent alignment, with no bowing or tilting to indicate that it is under stress. The concrete is in reasonably good condition considering the age of the structure and the environmental conditions.

Under excellent visibility conditions, the below water investigation revealed no problem areas in the boat basin. The steel sheeting alignment was good, and all joints were secure.

4.5.3. Structural Analysis

The structural analysis was performed for the inner harbor structures to evaluate the maximum allowable live and vehicular loadings. The record construction drawings and the current structural condition were utilized to develop parameters for the analysis. For purposes of analysis, it was assumed that the soil behind the boat basin walls has a unit weight of 125 lbs per cu ft, at an angle of internal friction of 30°. These values are representative of sand and cohesionless material. The tierod was assumed to have a yield strength of 20,000 lbs per sq in.

The boat basin wall was analyzed for an overturning failure mode. The presence of deep pile supports indicate that failure due to sliding is unlikely. The analysis included passive and active earth pressures. Furthermore, a surcharge load on the roadway adjacent to the boat basin was analyzed to estimate allowable live and vehicular loads.

Analysis results indicate that the basin wall will remain stable under a surcharge loading of 300 lbs per sq ft. For live loading and vehicles with a weight distribution of 300 lbs per square ft of vehicle area should be acceptable for the roadway adjacent to the boat basin.

4.5.4. Remedial Measures

The steel sheet pile wall around the perimeter of the boat basin is in good condition. The concrete wearing surface adjacent to this wall is beginning to deteriorate in areas, but this is of little consequence in terms of structural integrity. Eventually, a new wearing surface will have to be cast on top of the existing one. Existing wood rails which surround the upper level of the concrete wall adjacent to the boat basin are in various states of disrepair. Approximately 50% of this wood railing is in need of replacement. If this railing is not replaced, the steel angle irons used to provide support should be removed for safety purposes. A steel pipe railing which extends around the south side of the boat basin is not stable in many areas due to the deteriorating nature of the retaining wall. Approximately 80% of this railing is in need of repair. This railing should be removed and replaced. As part of this replacement, the concrete retaining wall, which has spalled for a majority of its length, should be repaired. This repair would consist of removal of the existing concrete down to sound material and recapping of the wall. It is estimated that approximately 1000 ft of wall will have to be recapped. The concrete spalling damage should be repaired under the remainder of the wall. The portion of the pipe railing to be repaired is approximately 600 ft in length. The opinion of probable cost to remediate the boat basin seawall is \$110,000 (\$32,100 material, \$77,900 labor).

When repaired, the estimated remaining useful life of the seawall is approximately 15 - 20 years. If a routine maintenance program is implemented, the useful life could be extended.

4.6 South Seawall

4.6.1. Description

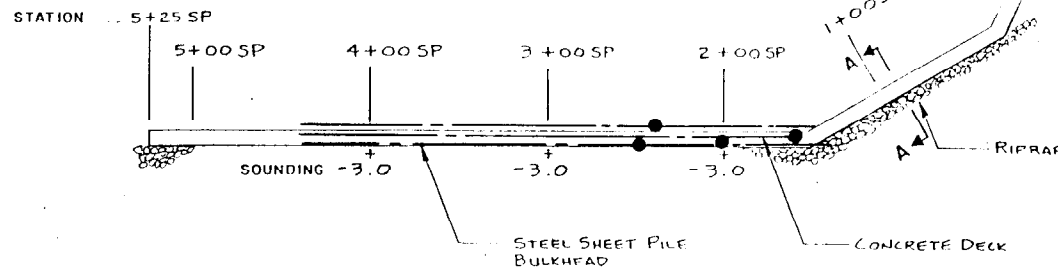
The south seawall is constructed of steel sheet pile. The piling extends approximately five feet above the waterline. The water depth varies between one and five feet. The imbedment depth and date of construction are reportedly not known and construction plans are not available. The piling extends approximately 525 feet along the harbor shoreline between the south pier and south breakwater. A timber wale has been attached just above the water line. Concrete rubble and riprap have been placed at the base of the piling over most of the seawall's length. The height of riprap varies from 0 to 4 feet.

A concrete deck is located adjacent to the seawall. Stationing of this structure begins at the north end and increases proceeding to the south. The letters "SP" follow the station to indicate the sheet pile structure. Figure 16 illustrates the plan view dimensions and stationing.

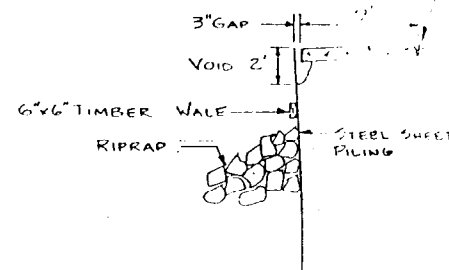
4.6.2. Visual Observations

The sheet piling appears to be in good condition with no visible signs of rust or deterioration. The timber wales are missing from station 1+90SP to 2+08SP and the steel connection rods are exposed. The sheet piles are visibly out of alignment between these stations. The alignment of the remainder of the structure is good (Photo 13.02).

It is possible that the wale connecting rods are part of a tieback system to support the sheet pile wall. A test pit excavation was performed with a backhoe at Station 0+95SP to investigate the possibility of tiebacks. The test pit was excavated eight feet west of the wall and adjacent to an observed connecting rod. The excavation was carried 5.5



PLAN



SECTION A-A

NOT TO SCALE

NOTE:

SOUNDINGS ARE GIVEN IN FEET BELOW NAVAL BASE DATUM OF 0.00'

NON-DESTRUCTIVE TESTING LOCATIONS:

SIR SURVEY
CONCRETE CORE



STS Consultants Ltd.
Consulting Engineers

FACILITY NO. 746
SOUTH SEAWALL PLAN & SECTION
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

(REFERENCE 1)

DRAWN BY G.R.S.	DATE 5-88	SCALE SHOWN	STS PROJECT NO. 1137-1
CHECKED BY L.M.B.	DATE 6-88	SHEET NO. 16	STS FILE NO.

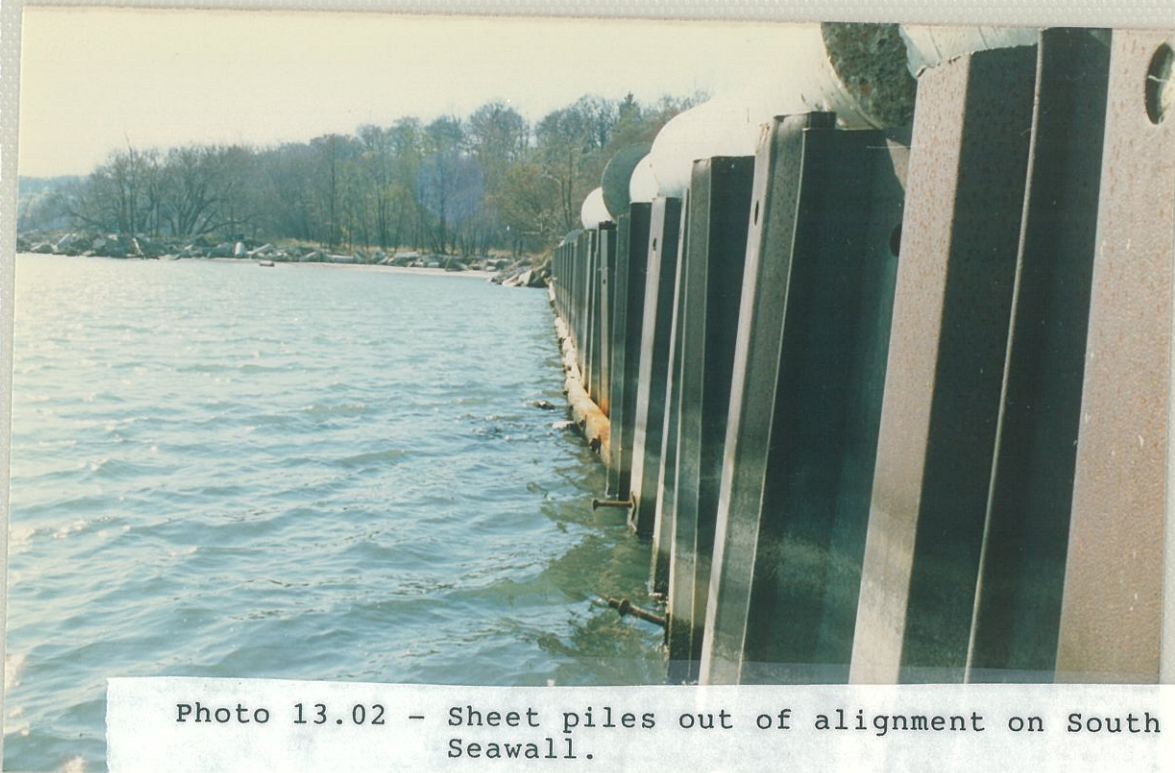


Photo 13.02 - Sheet piles out of alignment on South Seawall.

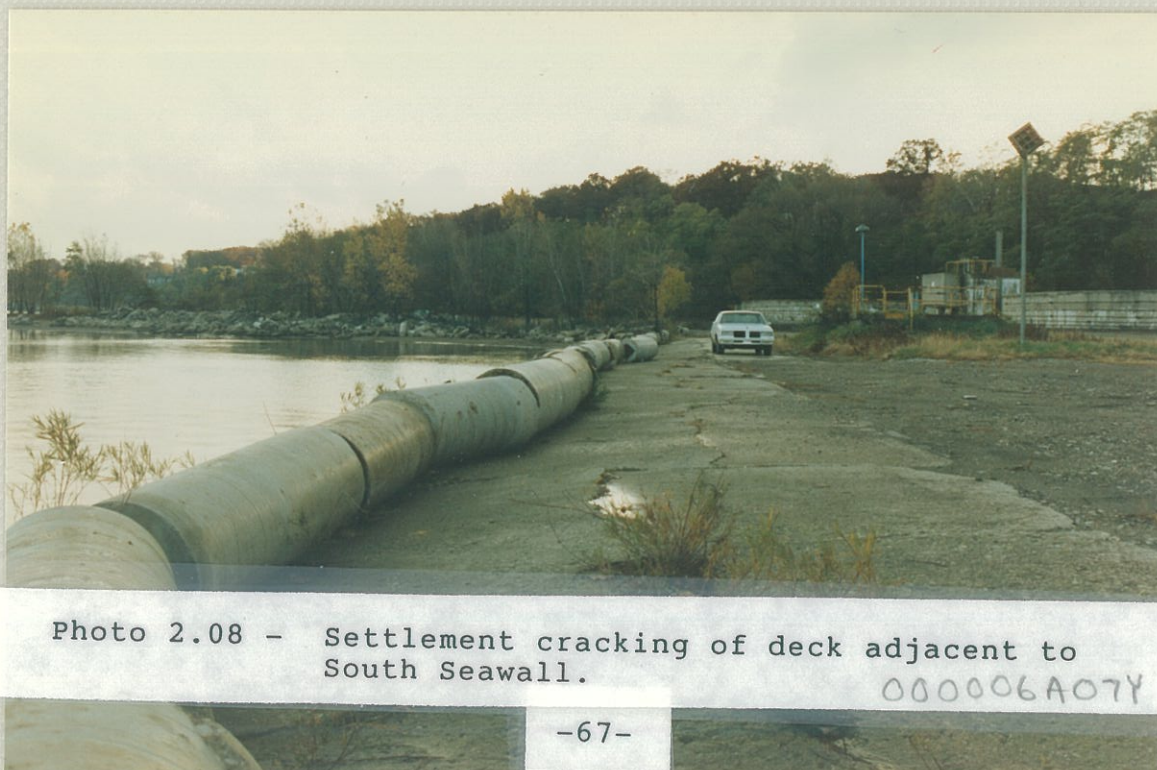


Photo 2.08 - Settlement cracking of deck adjacent to South Seawall.

000006A07Y

feet below grade, one foot deeper than the tieback would be expected. No tieback or anchor system was found. Therefore, it is assumed that the wall acts as a cantilever system.

The concrete deck adjacent to the seawall has a crack traversing the centerline, and tilts toward the sheet piling (Photo 2.08). The deflection is great enough that the piling extends approximately eight inches above the concrete deck in some locations. Subsurface Interface Radar (SIR) exploration was used to investigate the possibility of voids under the deck caused by leakage of support soils through the sheet pile joints. A detailed discussion of this exploration methodology is presented in Section 3.0.

Large concrete cylinders three feet in diameter and three feet long have been placed adjacent to most of the piling to form a safety barrier. These cylinders appear to be demolition debris.

Most of the the timber wales are badly damaged and deteriorated. The connecting bolts are no longer tightened against the sheet piling. Missing wales were observed at Station Nos. 1+50SP, 2+00SP, and 2+75SP. Concrete rubble riprap has been placed along portions of the seawall. This has increased the stability of the wall in areas. Sheet piles were in good condition, with no noticable damage.

4.6.3. Non-Destructive Testing

A subsurface interface radar survey was performed across the concrete adjacent to the steel sheet pile seawall. The objective of this survey was to explore for the potential presence of voids beneath the concrete slab. The radar survey indicated the presence of potential voids very near the seawall, and no voids of significance away from the seawall. Two concrete cores were obtained to verify the results of the radar survey. Core No. 7 was conducted approximately one foot from the edge of the sheet piling.

Four inches of free space was observed between the bottom of the slab and the underlying sands and gravels at this location. No void was observed at Core No. 8 which was taken a further distance from the seawall. No reinforcement bars were encountered in the cores. Voids present near the seawall are consistent with what was found in the 1980 Inspection Report. Movement of the steel wall towards the water over time has resulted in shifting and cracking of the concrete. Rainwater which gets into the ground behind the seawall has leached some of the soil materials thereby forming the void.

Steel sheet piling thickness measurements were taken at 61 locations. Measurements were obtained along the full depth of the sheet pile from the waterline up to the structure, and along its entire length. The thickness measurements ranged between .36 and .42 in. Although no construction plans are available, it appears that the steel sheeting is a PDA-27 section or greater. The PDA section would have an original thickness of .375 in. which is equal to or exceeded in almost every measurement that was obtained. The thickness measurements verify the visual observations that the steel wall has not undergone any significant deterioration.

4.6.4. Structural Analysis

There are no record drawings for this steel sheet pile bulkhead in the South Shore Protection Area. Therefore, the length of the steel sheeting is unknown. However, the alignment of the seawall appears to have stabilized. Movement which has occurred appears to be associated with damage to a timber wale that used to supply lateral support between the sheets. Although no structural analysis could be performed to assess the uniform or live loads that are allowable for this section, it appears that a uniform load of 300 lbs per sq ft would not be a problem. This conclusion is based upon the fact that a good amount of concrete rubble has been placed on the water side of the structure. Furthermore, the structure has been in-place for many years and no serious damage has occurred to date. It does not appear that this section of wall is subjected

to excessive surcharge loads. If this changes in the future, it is recommended that the wall be repaired to include the filling of voids, recapping of the concrete deck, and placement of additional rubble on the lakeside of the wall. The wall should be monitored to ensure that scour does not occur causing undermining in the future. It would make sense to keep all vehicular and excessive surcharge loads a safe distance from the wall. A distance of 15 ft is suggested. This also makes sense from a safety standpoint since there is no formal guardrail at the edge of the seawall.

4.6.5. Remedial Measures

Although this section of seawall does not appear to be a critical section in terms of waterfront operations, it is recommended that the same repairs suggested in the 1980 report be implemented. This would include filling of potholes and voids adjacent to the wall, and recapping of the concrete deck with pavement. This capping should be accomplished in such a way that drainage water does not enter behind the seawall thereby exacerbating the formation of voids. The opinion of probable cost to accomplish this repair is \$5,300 (\$4,000 material, \$1,300 labor). With this repair implemented, the remaining useful life of the wall would be approximately 20 years or more. Please note that this wall should be monitored regularly to observe potential scour action on the lakeside of the wall.

4.7 Boat Ramp

4.7.1. Description

There are two boat ramps located near Building 51 adjacent to the North Pier. The ramps slope into the outer harbor and are constructed of precast paving blocks laid side by side. Timber docks lie on either side of each ramp as illustrated in Figure 17.

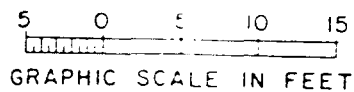
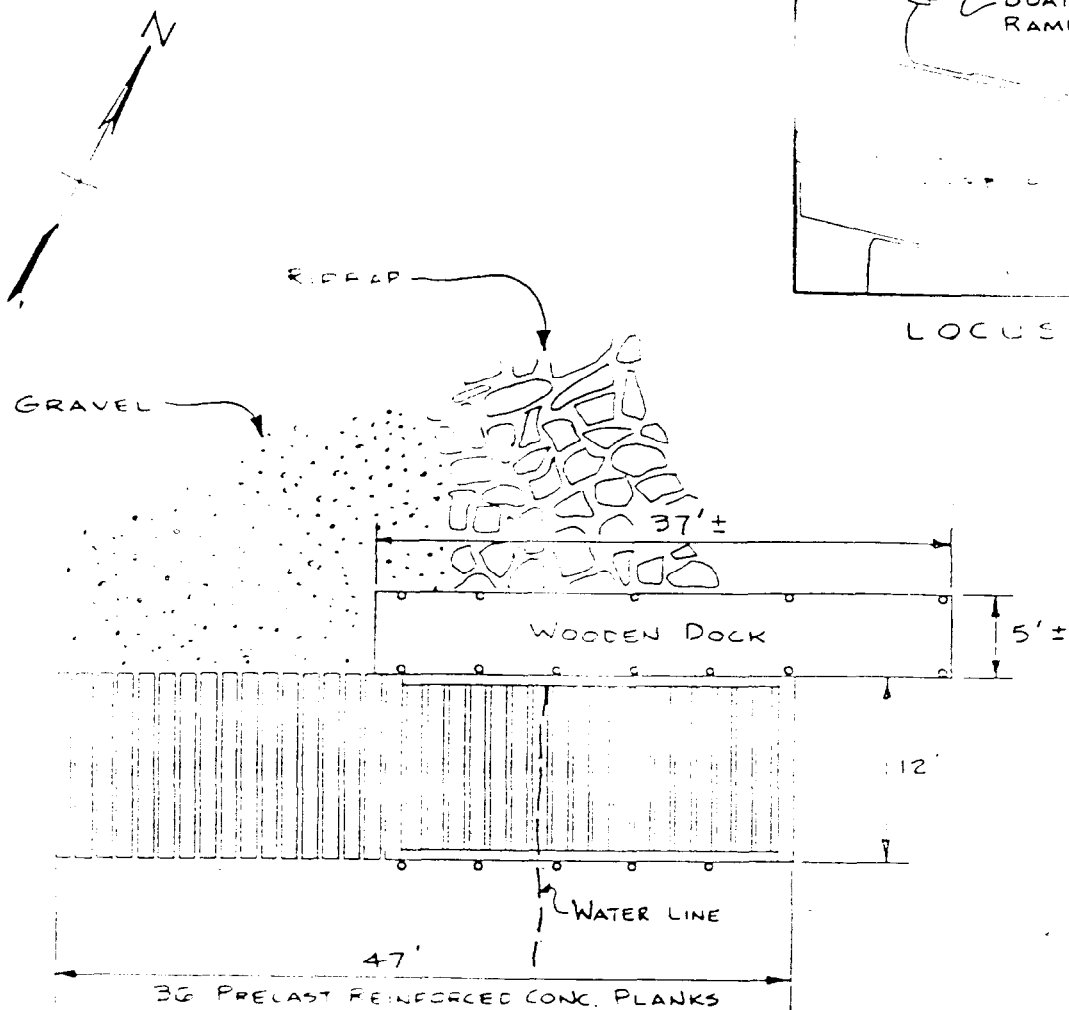
4.7.2. Visual Observations

The above water portions of the boat ramps appear to be in good condition. No obvious movement or deflection of the ramps was observed. The above water portion of the piers was in good condition with no signs of deterioration or damage.

Some slight breaks were observed in the concrete slabs in the underwater portion of the ramps, but the damage was not thought to be severe. The piers supporting the docks were found to be in good condition without damage or scour.

4.7.3. Remedial Measures

No repairs are required for this structure at this time. The remaining useful life should be 15 to 20 years.



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PROJECT/CLIENT

FACILITY NO. 721
BOAT RAMP
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

DRAWN BY G.R.S. 5-88

CHECKED BY L.M.B. 6-88

APPROVED BY

SCALE SHOWN FIGURE NO. 17

STS DRAWING NO. 1137-1

4.8 North Seawall

4.8.1. Description

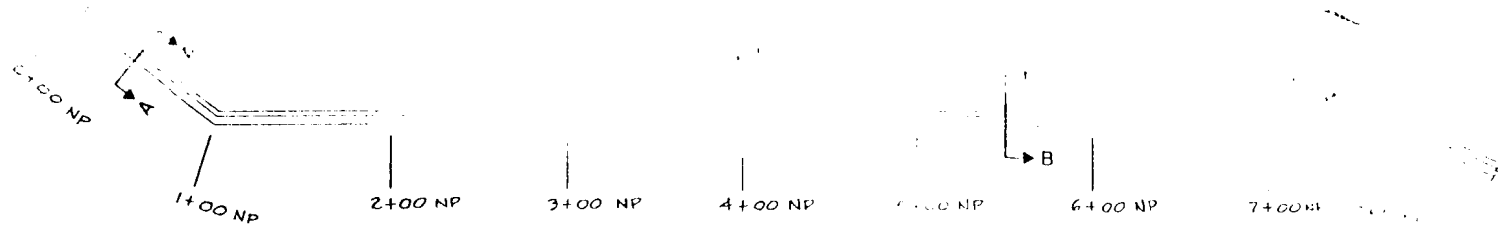
The north seawall is a concrete structure extending for 1450 feet as illustrated on Figure 18. This wall has six different cross section configurations as illustrated on Figure 19. The standard shape of the seawall is a gravity retaining wall approximately eight feet tall, with a walkway about 2.5 ft below the top of structure. At various locations, the face below the walkway is either vertical, sloped, or stepped. The top of the wall has an overhang shape. No record drawings are available for the structure; therefore, its age and original alignment are unknown. The structure appears to have been modified several times since its original construction.

The north seawall was stationed beginning at the south end with stations increasing to the north. The letters "NP" after the station designate the north seawall structure.

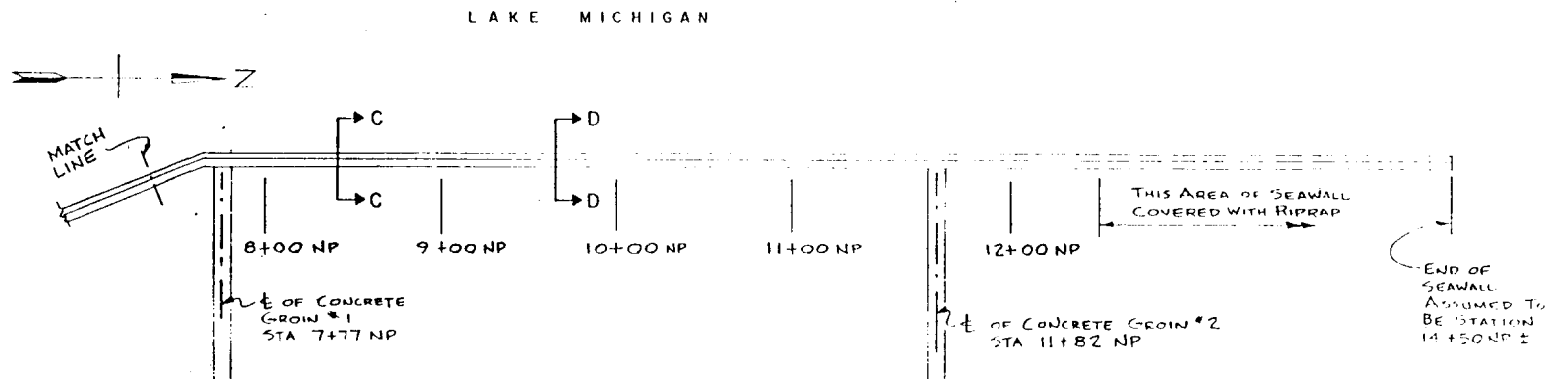
4.8.2. Visual Observations

The section of the seawall between 0+00NP and 1+00NP is protected by deposited beach sand which is trapped by the south breakwater. Concrete rubble has been placed on the sand against the base of the seawall. This section shows evidence of spalling along the overhang, walkway, and concrete edges, but the loss is less than two inches and no reinforcing steel is exposed.

From Station 1+00NP to 2+00NP, the lakeshore is located about 10 feet off the toe of the structure. In this location, the base has been protected by placing concrete cylinders approximately 3 feet in diameter and 3 feet long in a random fashion between the toe of

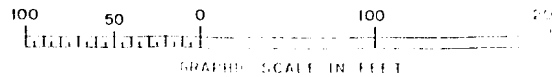


PLAN



PLAN

LAKE MICHIGAN

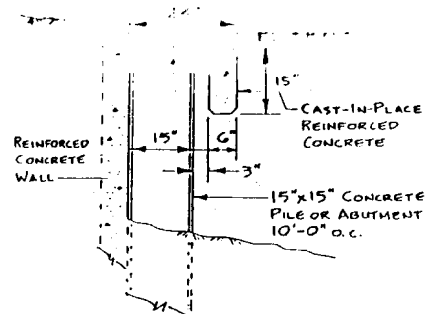


STS Consultants Ltd.
Consulting Engineers

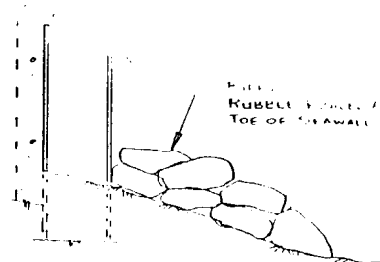
FACILITY NO. 722
NORTH SEAWALL PLAN
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

(REFERENCE 1)

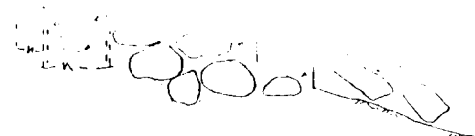
DRAWN BY	DATE	SCALE	STS PROJECT NO.
G.R.S.	5-88	SHOWN	1137-1
CHECKED BY	DATE	SHEET NO.	STS FILE NO.
L.M.B.	6-88	18	



ORIGINAL CONSTRUCTION (ASSUMED)

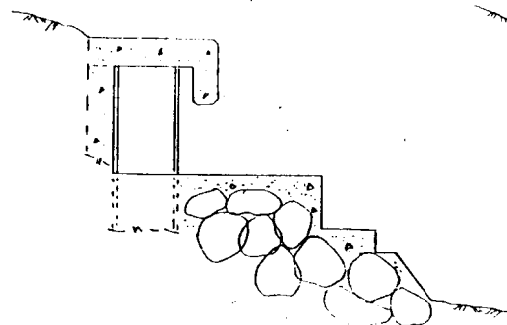


PHASE II CONSTRUCTION



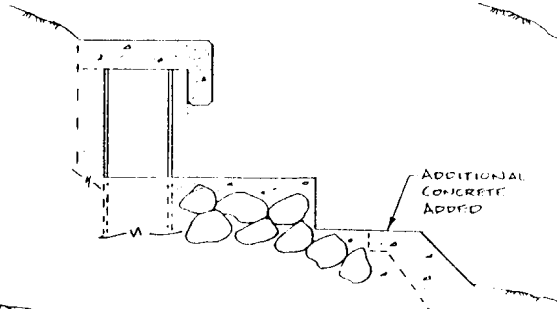
SECTION A-A

CEL STA 0+00NP TO STA 5+50NP
SIMILAR



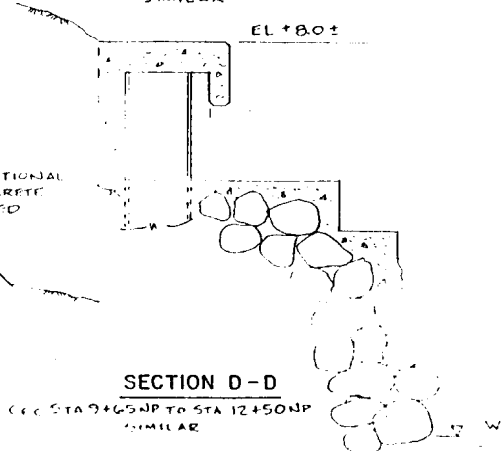
SECTION B-B

CEL STA 5+50NP TO STA 8+40NP
SIMILAR



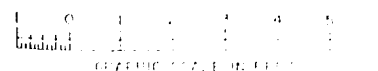
SECTION C-C

CEL STA 8+40NP TO STA 9+65NP
SIMILAR



SECTION D-D

CEL STA 9+65NP TO STA 12+50NP
SIMILAR



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FACILITY NO. 722
NORTH SEAWALL SECTIONS
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS

(REFERENCE 1)

DRAWN BY	DATE	SCALE	STS PROJECT NO.
G.R.S.	5-88	SHOWN	1137-1
CHECKED BY	DATE	SHEET NO.	STS FILE NO.
L.M.B.	6-88	19	

structure and the water's edge. The concrete is placed to the elevation of the walkway. Minor surface spalling of one to two inches has also occurred in this section on the walkway surface and concrete edges.

A freeway-type guardrail was constructed over the overhang section from Station 2+00NP to 4+40NP. The guardrail is supported by reinforced columns on 10 foot centers. The supports from 2+20NP to 2+40NP have been damaged, and are no longer carrying weight. They are only attached to the structure through the reinforcing steel. The guardrail shows evidence of having been struck, which explains the damage to the columns. New supports have recently been constructed from Stations 2+50 to 3+10.

The condition of the concrete structure between Stations 2+00NP and 5+50NP is equal to the previous section. The new concrete guardrail supports are in excellent condition. The base of the seawall is protected by large concrete rubble, which extends to the walkway level from Station 2+00NP to 3+10NP, and to the top of structure from 3+10NP to 5+50NP.

Beginning at 5+50NP through 11+80NP, no stone protection has been placed at the base of the seawall. A sand beach extends about 15-20 feet from the seawall toe to the Lake Michigan shoreline. The concrete in this section is not in as good a condition as in the previous sections. Locally severe spalling is evident near construction joints to a depth of three to seven inches. In several locations, major spalling of entire sections of the overhang has occurred and reinforcing steel is exposed. Major overhang spalling has occurred at the following Stations: 5+50NP, 5+90NP, 6+10NP, 6+50NP, 7+80NP through 8+10NP, and 8+55NP through 9+00NP. Approximately, 150 ft of this overhang is in need of immediate repairs (Photo 12.22).

From 11+80NP to 12+50NP, the entire seawall has been backfilled with concrete rubble giving the appearance of a riprap revetment rather than a concrete seawall. Small trees



Photo 12.22 - Severe spalling of North Seawall.

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and other vegetation grow from the crack between the seawall and Ziegemeyer Road. The remaining portion of the seawall has been landfilled by construction of the riprap groins and placement of fill material.

Large trees are growing adjacent to or through the structure at stations 2+00NP, 3+70NP, 5+50NP, 7+70NP, and 11+00NP. Four holes were found adjacent to the structure. The first is located at 6+70NP, is about two foot by two foot by one foot deep, and could indicate either a drainage or settlement problem. It has been marked by NTC personnel with a blue paint arrow. The second hole is located at 6+98NP, about three feet by three feet by 18 inches deep. The third hole, located at 7+20NP, is two feet by one foot by two feet deep and resembles an animal burrow. At the north end of the structure, at station 15+15NP, a manhole is located which outlets to two 15 inch cast iron pipes which run out along the centerline of the nearby riprap groin. The fourth hole is located over the centerline of this storm sewer, and is about two foot cubed in dimension. This hole has also been marked with a blue arrow.

There is a 100 ft stretch of seawall located approximately between Station Nos. 10+10NP and 11+00NP which is being subjected to undermining. There is no concrete apron or rubble protection in front of this portion of the structure. One 10 ft stretch of wall in this area has already been undermined and is in need of repair. The void at this location extends underground for a distance of 5 to 8 ft.

4.8.3. Remedial Measures

In general, the condition of the concrete seawall appears satisfactory. The portion of the seawall which has been buried in rubble is in satisfactory condition. Approximately 10% of the remainder of the upper portions of the concrete wall are in need of repair. The toe of structure along the entire 1450 ft of lake shoreline is in reasonable

Department of the Navy
STS Project No. 1137-I
July 29, 1988

condition, with the exception of a 100 ft reach located approximately between Station Nos. 10+00 and 11+00. In this reach, a rubble mound revetment should be placed in front of the structure. The opinion of probable cost for this repair is \$35,000 (\$12,000 material, \$23,000 labor).

Repair of the undermined section of the concrete wall near the steel tanks located approximately at Station 11+00NP can be accomplished by the placement of concrete under the structure and a rubble mound revetment in front. The opinion of probable cost for this repair would be \$1,000 (\$300 material, \$700 labor). Finally, the upper portions of the concrete seawall are deteriorating for approximately 15% of the wall's length. In many areas, the concrete has deteriorated to the point where the steel reinforcement is completely exposed. No record drawings are available for this portion of the structure; therefore, it is impossible to determine the structural integrity of this section. The deteriorated portions of the wall should therefore be repaired back to their original condition. For ease of construction, it would probably make sense to chip away the poor areas of concrete and cap the top of the structure with a block of concrete. The opinion of probable cost for this repair would be approximately \$11,000 (\$3,000 material, \$8,000 labor).

4.9 Concrete and Timber Groins

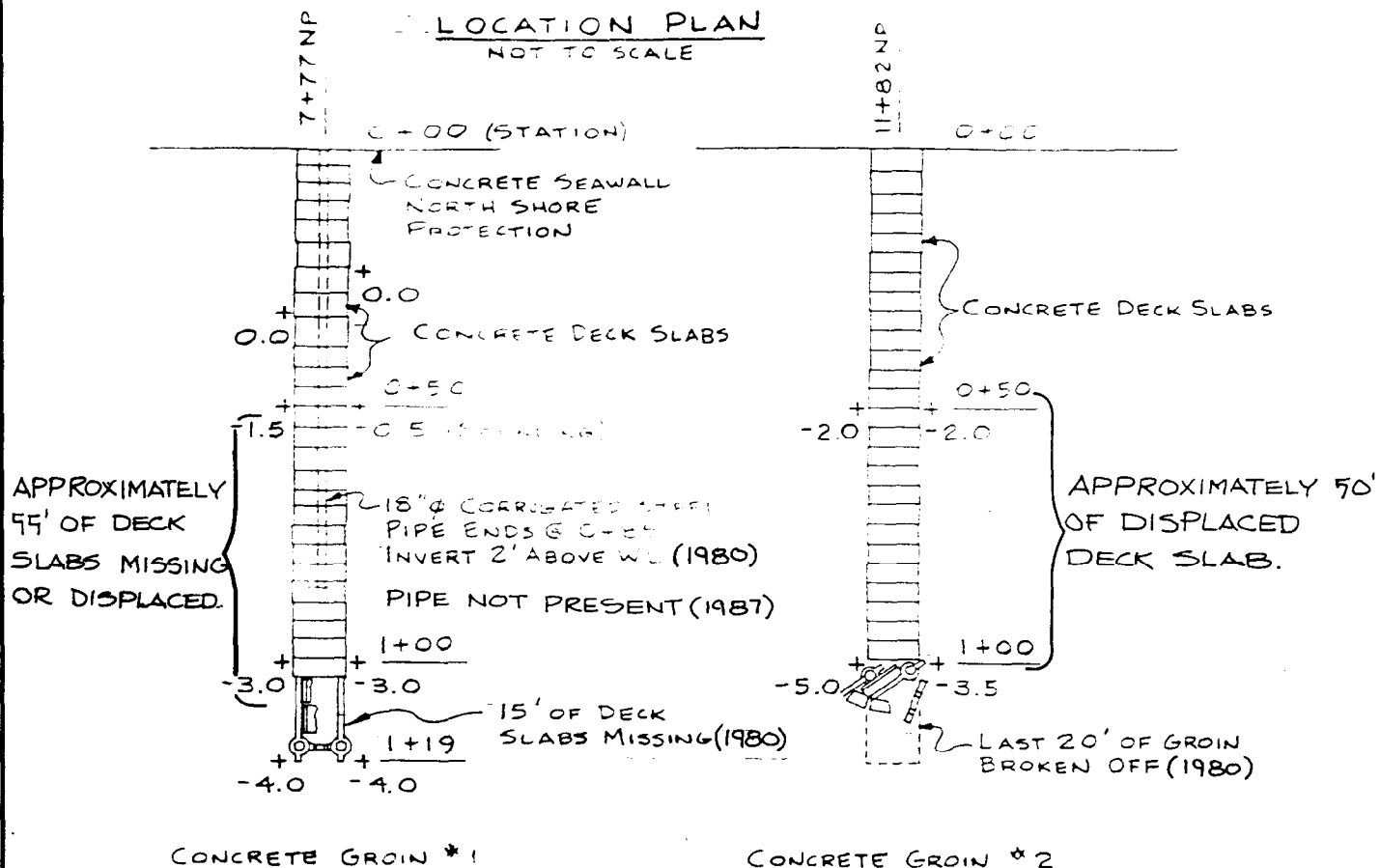
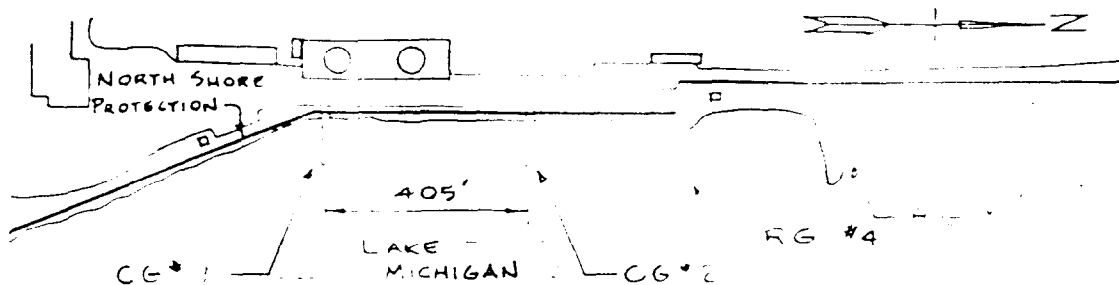
4.9.1. Description

There are two concrete groins located perpendicular to the concrete seawall described in Chapter 4.8. These groins are located perpendicular to the shoreline at Stations Nos. 7+77NP and 11+82NP. Both groins extend a distance of approximately 119 ft offshore as illustrated on Figure 18. The groin designated CG #1 used to service as support for an 18 inch circular corrugated steel pipe. The original purpose for these groins are unknown; however, they may serve to withhold some beach materials during periods of low water. Figure 20 illustrates the configuration of these groins.

Two additional groins constructed of timber piles are located approximately 0.7 miles north of the concrete groins on a portion of the shoreline adjacent to the FBI rifle range. These groins extend from 90 to 150 ft offshore and were constructed with two closely spaced parallel rows of timber piles approximately 8 ft apart. Figure 21 illustrates the configuration of these groins.

4.9.2. Visual Observations

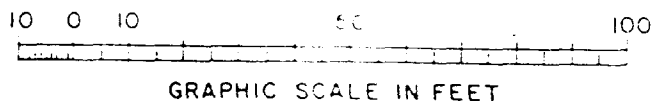
The precast concrete blocks have experienced large movement and displacement due to wave action. The groin has a circular cutout on the interior, and was probably used as protection for an outfall pipe. No pipe is visible in either groin at present. The groin structure is permeable due to the configuration of the blocks. The structure has been ineffective at trapping littoral drift, and minimally effective at dissipating wave energy (Photo 12.13). Both groins have failed in terms of structural integrity and performance.



PLAN

NOTE:

SOUNDINGS ARE GIVEN
IN FEET BELOW NAVAL
BASE DATUM OF 0.00'.



STS Consultants Ltd.
Consulting Engineers

PROJECT/CLIENT

FACILITY NO. 724-725
CONCRETE GROINS
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

DRAWN BY

G.R.S. 5-88

CHECKED BY

L.M.B. 6-88

APPROVED BY

SCALE

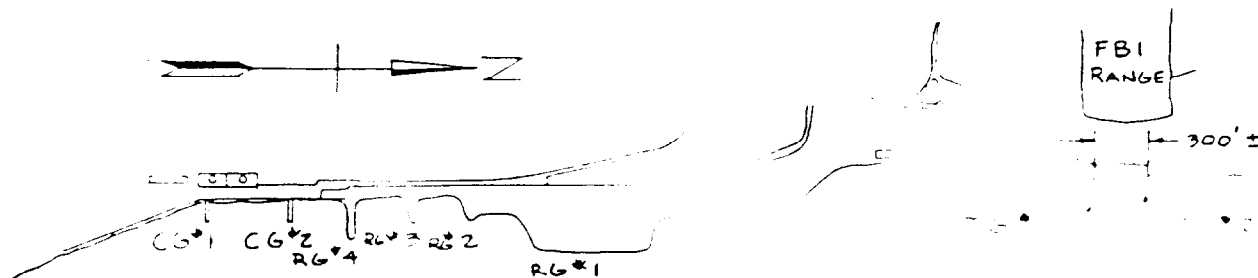
SHOWN

FIGURE NO

20

STS DRAWING NO.

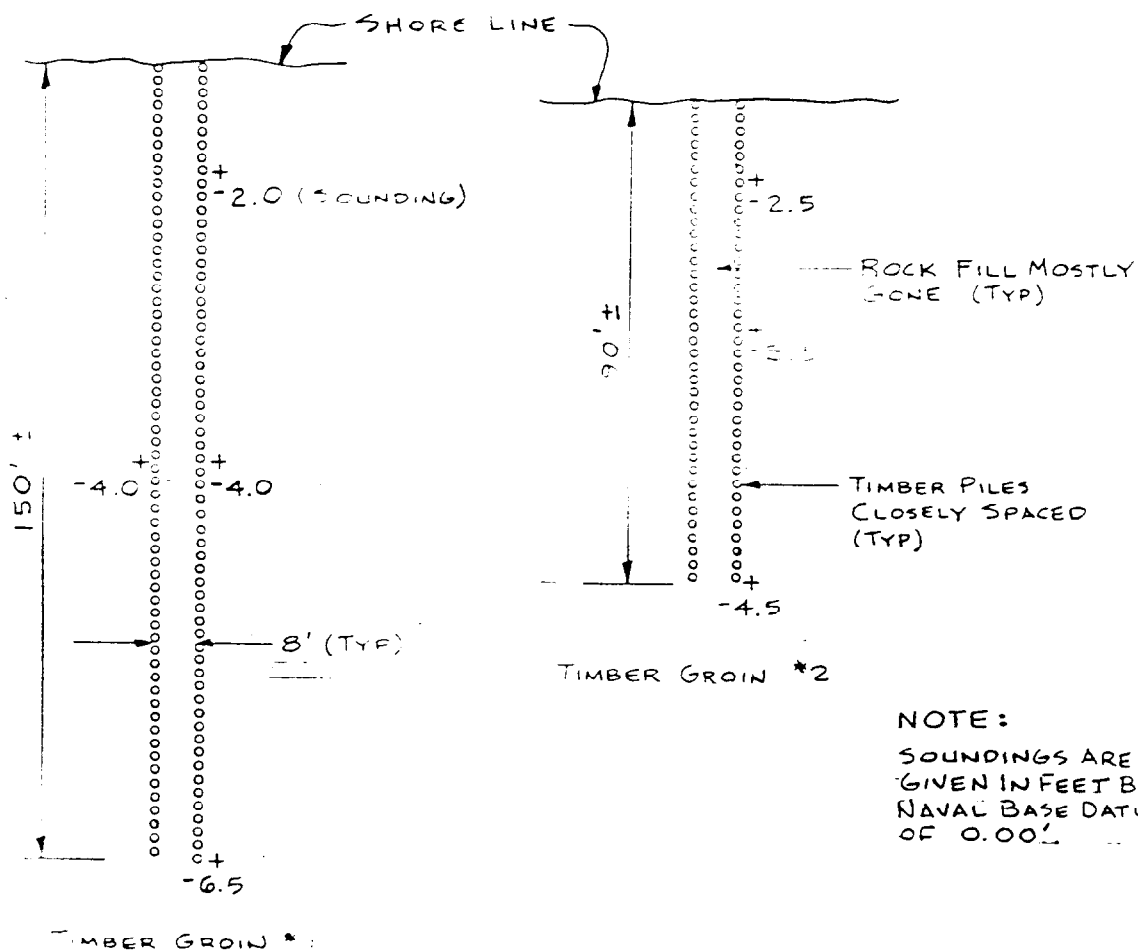
1137-1



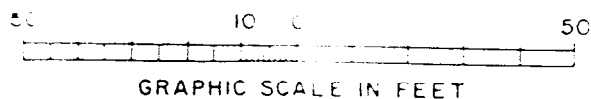
LAKE MICHIGAN

LOCATION PLAN

NOT TO SCALE



PLAN



STS Consultants Ltd.
Consulting Engineers

PROJECT/CLIENT

FACILITY NO. 726-727
TIMBER GROINS
WATERFRONT FACILITIES INSPECTION
NAVAL TRAINING CENTER
GREAT LAKES, ILLINOIS
(REFERENCE 1)

DRAWN BY

G.R.S.

5-88

CHECKED BY

L.M.B.

6-88

APPROVED BY

SCALE

SHOWN

FIGURE NO

21

STS DRAWING NO.

1137-1



All that remains of the timber groins are two pairs of closely-spaced piles. The structures have failed in terms of structural integrity and performance.

4.9.3. Remedial Measures

Both concrete groins appear to serve little or no function. Therefore, repairs are not warranted. The same conclusion applies to the failed timber groins located adjacent to the FBI rifle range. The slope adjacent to the timber groins is protected from erosion by a rubble revetment that was bulldozed from the top of the slope. In the short term, this shore protection is performing satisfactorily; however, the erosion protection is hap hazard at best and will require constant attention. As discussed in the NTC high water level study (Reference 2), remedial repairs will be required at the toe of slope to protect against slope failure. Any erosion of the existing shoreline in this area should be addressed as it occurs to guard against toe failure of the slope which could result in potentially catastrophic bluff failures.

There is some question as to the basic stability of the existing slope in this area (Reference 3). Additional slope failure analyses should be performed to evaluate the potential for bluff failure at this location. Flattening of the slope (2.5 horizontal min.: 1 vertical) will probably be necessary. The opinion of probable cost to flatten the slope and to construct a proper revetment is \$296,000 (\$186,700 material, \$109,300 labor).

4.10 Riprap Groins

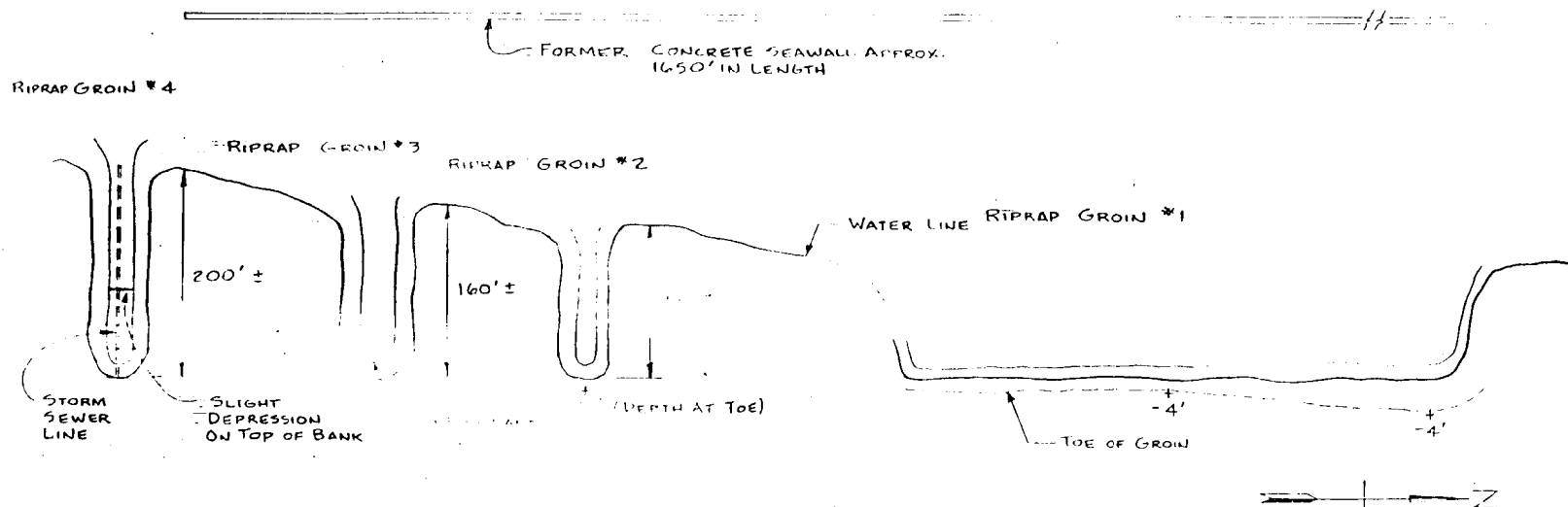
4.10.1 Description

Four rubble riprap groins were constructed as part of a land reclamation project in the mid 1970's. As part of this project, the ends of the groins were to be connected, and the interior section filled, covered with topsoil, and seeded. Phases I and III of the project have been completed. Phases II, IV, and V called for filling the spaces between existing groin nos. 2, 3, and 4, and have not been completed. Figure 22 illustrates the configuration and dimensions of this area. A portion of the former concrete seawall is buried landward and perpendicular to these groins.

4.10.2 Visual Observations

The lakefill area is protected by a rubble mound revetment constructed of concrete demolition materials (primarily slabs). Along the northern revetment (north of riprap groin No. 1), the concrete ranges in size from 0.5 to two feet thick and two to six feet in diameter. The face of the revetment is sloped at about one vertical to one horizontal. The revetment top is about 15 feet wide, approximately eight feet above normal water and about one foot above the lakefill. Erosion of topsoil has occurred at the edge of the lakefill, leaving a void about two to three feet wide between the lawn and rubble areas. The void is six to twelve inches deep at the northern revetment end, increasing to two to three feet at the southern end. Pea gravel is visible at the bottom of the void, which tends to collect driftwood and other debris (Photo 13.05)

The remainder of the riprap groin area designated Nos. 2, 3, and 4, is protected by smaller sized rubble. The rubble slabs in this area are six to eight inches thick, and about three feet in diameter, and have been placed in an armor-plate fashion. The slabs are parallel to the wave action direction. Additional protection appears to have been placed recently in the bay between groin nos. 2 and 3.



NOTES:
SOUNDINGS ARE GIVEN
IN FEET BELOW MLLW
BASE DATUM OF 0.00

PLAN



		STS Consultants Ltd. Consulting Engineers	
		FACILITY NO. 723 RIPRAP GROINS WATERFRONT FACILITIES INSPECTION NAVAL TRAINING CENTER GREAT LAKES, ILLINOIS	
(REFERENCE 1)			
DRAWN BY G.R.S.	DATE 5-88	SCALE SHOWN	STS PROJECT NO 1137-1
CHECKED BY L.M.B.	DATE 6-88	SHEET NO 22	STS FILE NO



Photo 13.05 - Erosion of landfill adjacent to Riprap Groin revetment.



Photo 13.19 - Erosion of topsoil on Riprap Groin No. 4.

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Erosion similar to that observed on the northern section is also occurring at the edge of the lawn along the remaining groins. On groin No. 3, the topsoil has been eroded away from the outer half of the structure. On groin No. 4, the middle section of the topsoil has been eroded, leaving an island of grass on the outer half of the structure (Photo 13.19).

4.10.3. Remedial Measures

The rubble revetment shore protection in Section 4.10 is in various states of disrepair. In general, the structure is performing well with only minor erosion problems noted. If the erosion of grassed areas that has occurred in the past is acceptable, no remedial measures will be required at this time. These problems can be expected to continue in the future. The minor erosion that has occurred could be fixed for a probable cost of \$100 to \$200 per lineal foot by the placement of a stone revetment with appropriate filter at the top of the existing rubble slope. Failures of the rubble shore protection can be expected from time to time due to the haphazard nature of construction. When these failures occur, they can be addressed by the construction of appropriately sized rubble revetments for a cost of \$200 to \$400 per lineal foot. There does not appear to be any immediate need for remedial measures at this time. The above estimated range of costs can be broken down to approximately 35% for materials and 65% for labor.

Department of the Navy
STS Project No. 1137-1
July 29, 1988

5.0 REFERENCES

1. Childs Engineering Corp., Waterfront Facilities Inspection at the Naval Base, Great Lakes, Illinois, Contract N62472-80-C-6164, November 1980, Medfield, Massachusetts.
2. STS Consultants, Ltd., Lake Michigan High Water Level Study, Draft Report, April 1988, Northbrook, Illinois.
3. STS Consultants, Ltd., Comprehensive Slope Stability and Erosion Study, Draft Report, May 1988, Northbrook, Illinois.
4. Great Lakes Naval Training Center, One-Foot Contour Interval Topographic Maps (Scale: 1" = 50'), Great Lakes, Illinois, 1986.



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